Development of a Freeze-Thaw Test for Design of Soil-Cement

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A laboratory test for evaluating the durability of stabilized fine-grained soils subjected to repeated freeze-thaw cycles has been developed. Test specimens (2-in. by 2-in. diameter) molded in Iowa State compaction apparatus are used. The specimens are kept frozen from top with free water available at bottom. Winter climatic conditions in Iowa roads are simulated taking into consideration field conditions, such as a freezing temperature, availability of free water, and a proper temperature gradient. Climatic data and freeze-thaw calculations showed that 10 cycles of freezing and thawing are sufficiently severe for testing base courses. The criteria were the unconfined compressive strength and the index of resistance to the effect of freezing, defined as the ratio of the unconfined compressive strength of the freeze-thaw specimen to that of the immersed control specimen.

Laboratory test results are correlated with those of field trial sections of soil-cement base courses. Based on the performance of the test sections in which the cement content was the variable, an adequate soil and cement combination was chosen. The laboratory test results of the soil-cement are proposed as tentative criteria for the design of soil-cement for base courses. From the results of one year's study of the experimental pavement sections, 7 percent cement seems to be adequate for making durable soil-cement, though standard tests warranted 11 percent cement admixture.

•ALL components of pavements for roads and airfields must resist the destructive effects of alternate cycles of freezing and thawing in climates having such seasonal changes. Some laboratory techniques have been developed to evaluate the durability of stabilized soils subjected to repeated freeze-thaw cycles. The two most widely used methods are AASHO Method T 136-57—ASTM Method D 560-57 and British Standard Test 1924:1957.

The AASHO-ASTM method covers procedures for determining the soil-cement brushing weight losses, moisture changes, and volume changes produced by repeated freezing and thawing of hardened soil-cement specimens (1). Soil-cement weight loss as a result of brushing after freeze-thaw cycles is usually the critical criterion affecting mix design. A frequent criticism of this method of test is that it is overly severe and does not simulate field conditions.

The British Standard test determines the change in unconfined compressive strength of specimens of stabilized fine-grained soil when subjected to cycles of freezing and thawing under specified conditions. The British test is not restricted to soil-cement and perhaps simulates field freeze-thaw conditions better than the brushing test.

The British freeze-thaw test has been modified in the soil research laboratory of the Iowa Engineering Experiment Station (4). The principal modification was the use

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Paper sponsored by Committee on Soil-Portland Cement Stabilization.

of 2- by 2-in. instead of 2- by 4-in. specimens. The field conditions simulated in the modified test were the same as stipulated by the British test, though climatic changes in Iowa were more severe than those in England. The British test conditions should therefore be modified to suit Iowa climate.

DEVELOPMENT OF THE FREEZE-THAW TEST PROCEDURE

The field conditions for deleterious frost action are a freezing temperature, a readily available source of water, a thermal gradient and cycles of freezing and thawing.

In the AASHO-ASTM test the specimen is frozen but, unlike in the field, the freezing temperature is all around the specimen. Hence the thermal gradient is between the interior and all exterior faces of the specimen, not between the top and bottom as in pavements. Furthermore, the freezing temperature and number of freeze-thaw cycles are not varied to simulate climatic changes. The specimen absorbs water during thawing, but during freezing the need of water for ice segregation is satisfied mainly by the redistribution of moisture inside the specimen. Another severe test condition which does not simulate a field condition is the brushing weight loss of the specimen.

The British freeze-thaw test appears to have been designed to approximate the worst conditions for freeze-thaw damage to stabilized pavement components in England; freezing is from the top down. A realistic freezing temperature, a temperature gradient, and a constant source of unfrozen water during freezing and thawing are used. The emphasis is placed on the strength loss of the freeze-thaw specimen as compared with that of the control (continually immersed) specimen. However, no provision is made in the test procedure for adapting the test to different climatic conditions.

The purpose of the test procedure developed in this report is, therefore, to simulate the field freeze-thaw conditions in Iowa.

Establishment of Freezer Temperature

It was decided to have the temperature inside the freezing cabinet equivalent to the average minimum air temperature in Iowa during the winter months, November to March. An analysis of available climatic data of the U. S. Weather Bureau (15) for the 130 observation stations in Iowa over the 8-yr period 1952 to 1960 showed the temperature to be 20 F.

Establishment of Temperature Inside Vacuum Flask

The thermal gradient between the top and bottom surfaces of a specimen in contact with free water governs the movement of water to the freezing zone. Inasmuch as the top of the specimen is subjected to freezing temperature corresponding to the minimum average air temperature in winter, the bottom of the specimen should be kept at a temperature corresponding to that at a specified depth in the pavement or in the underlying soil. An analysis was made of available soil temperature data for Iowa (15) from the five observation stations over the 10-yr period 1950 to 1960. The maximum temperatures at depths of 1, 4, and 20 in. were averaged; the value obtained was 35 F. The overall effect of averaging the temperatures at depths up to 20 in. was to exaggerate the temperature gradient simulated in the 2-in. high test specimen.

Evaluation of Number of Cycles

Strength properties of a test specimen in the laboratory or the load-carrying capacity of a pavement component may be expected to vary appreciably with repeated freezing and thawing. In the thawed state the soil may lose as much as 80 percent of the strength it had prior to freezing (14). Hence thawing represents the critical period for a road base or subbase. Also, a single freezing may cause sufficient ice segregation to reduce the density and increase the porosity so that subsequent frost action effects become more intense. Therefore the load-bearing capacity is greatly influenced by the number and extent of freezing and thawing cycles.

The determination of the number of freeze-thaw cycles was based on a study of the daily maximum and minimum air temperatures obtained from Iowa weather records.

Daily winter temperatures at the 130 observation stations were averaged for a 2-yr period (1958-1960). From the daily maximum and minimum air temperatures, two graphs were plotted (Fig. 1). The median of the two graphs determined the number of cycles of freezing and thawing.

The frost penetration depths (Figs. 2 and 3) are based on the modified Berggren formula (7). The surface of a pavement would experience about 10 cycles of freezing and thawing and the top of the base might experience about 9 cycles in a single winter season.

The depth of freezing and the number of cycles of freezing and thawing have been verified in the field by installing thermocouples under a soil-cement base in Iowa primary highway 37. The results verify the theoretical calculations, in respect to the depth of freezing (Fig. 4). However, the surface experienced relatively few cycles of freezing and thawing. Therefore, the use of 10 cycles of freezing and thawing in the Iowa freeze-thaw test appears sufficiently severe.

Establishment of Thawing Temperature

Rapid thawing of a pavement component to shallow depths produces the most unfavorable condition of supersaturation above a residual layer. Thus, the rate and depth of thawing should be a major factor governing the strength of the material subjected to thawing. Therefore, the evaluation of a thawing temperature suitable to Iowa climate was significant.

Approximate calculations showed that a thawing temperature of 40 F was probably a satisfactory value to use in the test. However, recognizing that complete melting during spring thaw takes place at rather elevated temperatures, a value of 77 F was proposed for the test procedure.

Criteria for Evaluation of Test Results

Unconfined Compressive Strength.—The Iowa freeze-thaw test criterion is that the soil-cement test specimens subjected to 10 cycles of freezing and thawing must have an unconfined compressive strength of minimum specified value.

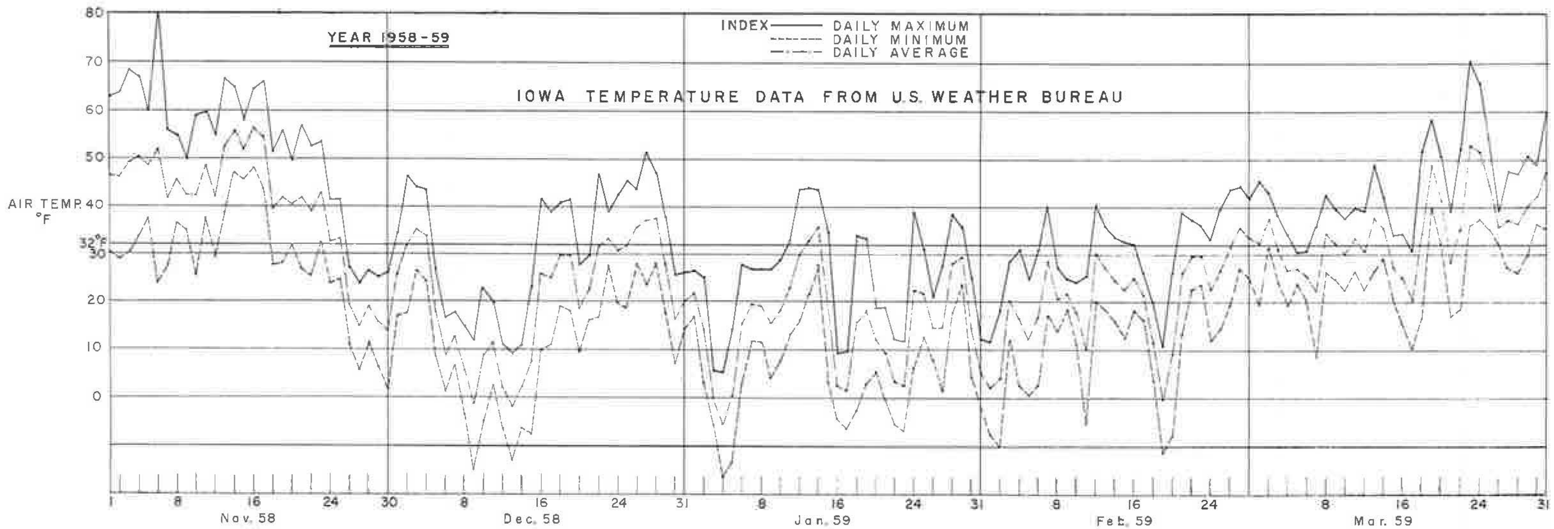
Index of Resistance to Effect of Freezing.—An important criterion of durability is the change in the unconfined compressive strength of test specimens subjected to freezing and thawing. The index of resistance to the effect of freezing is defined by the ratio of the unconfined compressive strength of the freeze-thaw specimen to that of the control specimen, expressed as a percentage. This criterion is a measure of the relative strength gaining capability of soil-cement during cycles of freezing and thawing.

Heave upon Freezing.—The question has often been asked as to how accurately the results of a laboratory freezing test indicate the frost behavior of a soil in the field, especially since the major problem is loss of strength on melting. According to Lambe (12), the rate of heave does give an indication of strength of thawed soil. The more the test specimen heaves, the greater the amount of water imbibed during freezing. The greater the quantity of water present on thawing, the lower the strength of the thawed soil. A subjective measure of heave may be the maximum heave expressed as a percentage of the initial height.

Moisture Content and Distribution After Tests.—Another criterion to evaluate the results of the Iowa freeze-thaw test may be the change in moisture content of the test specimens. The moisture content of soil at the beginning of freezing largely determines the amount of segregated ice and amount of heaving of the soil on freezing. The moisture content of the test specimens before and after testing and its distribution through the height of the specimen after cycles of freezing and thawing are shown in Figure 5. The soil is a friable loess. On the basis of these data, it may be concluded that the increase in the amount of soil moisture and its distribution through the soil seems to be a promising criterion warranting further investigation.

DEVELOPMENT OF STRENGTH CRITERIA

This part of the research was intended to relate pavement performance to the cement content of the soil-cement base in the field and thus develop criteria for soil-cement design.



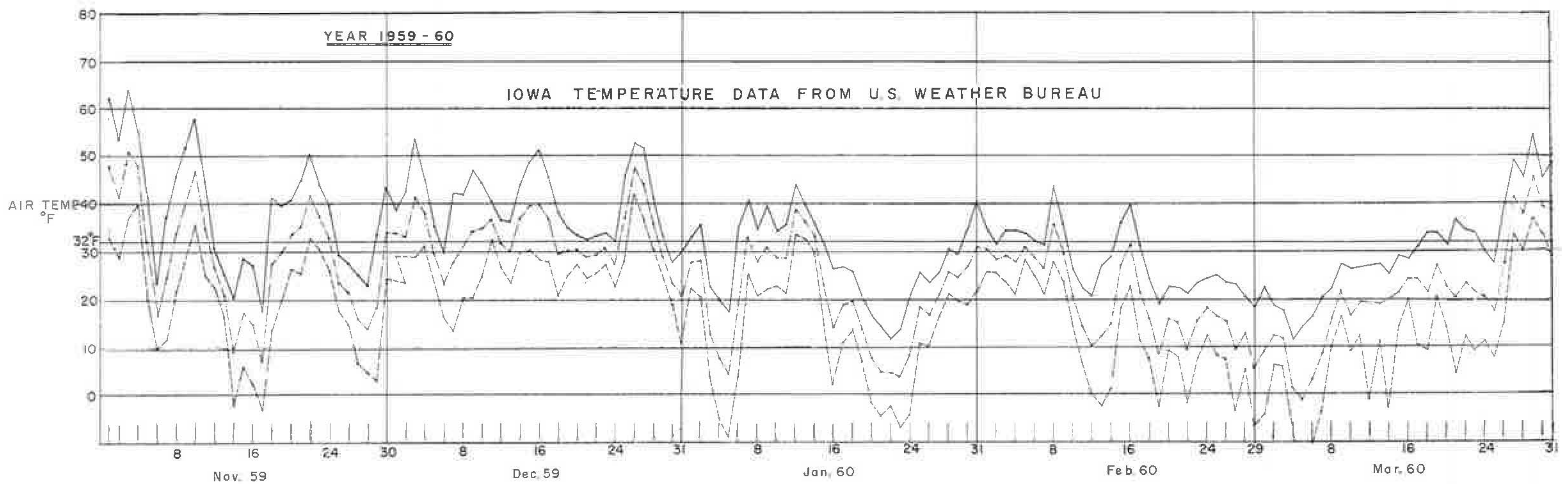


Figure 1. Average U. S. Weather Bureau 1958-1960 temperature data from 130 stations in Iowa.

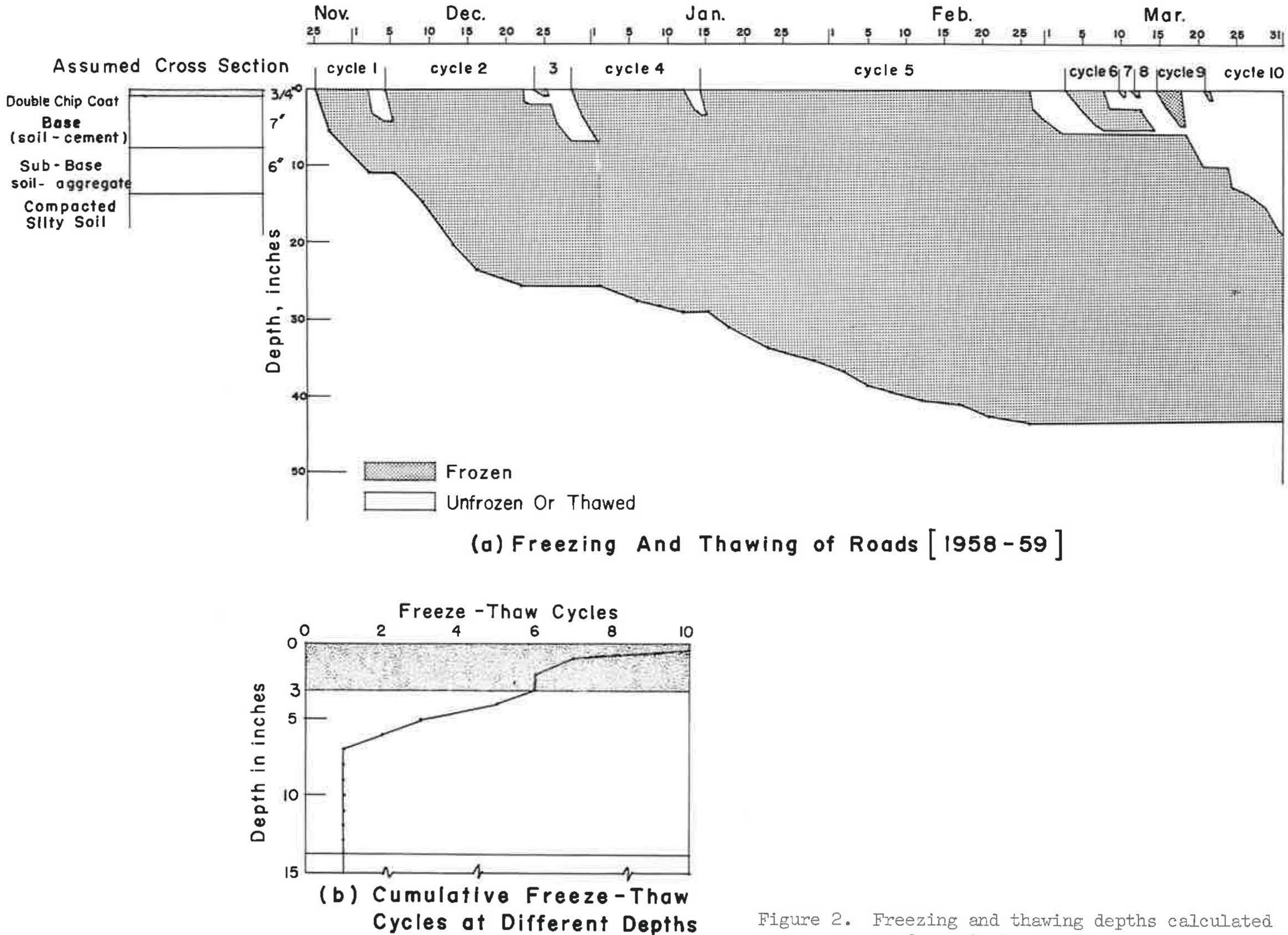
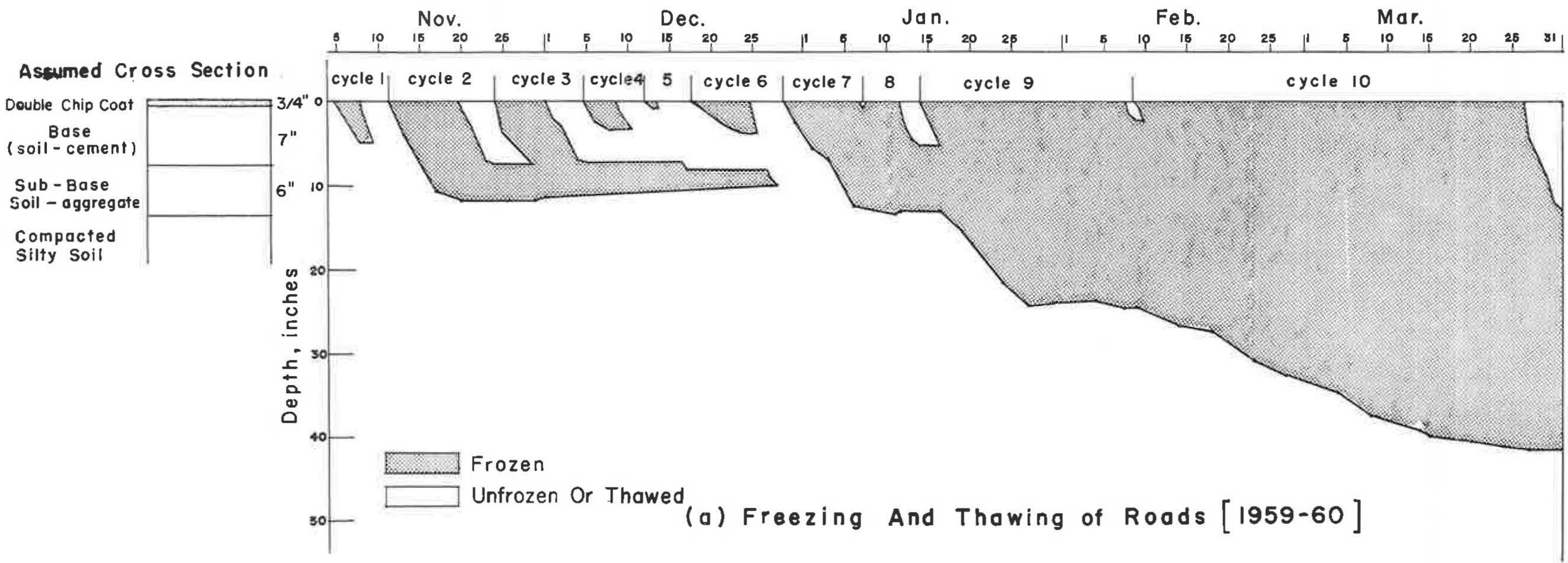


Figure 2. Freezing and thawing depths calculated from air temperature.



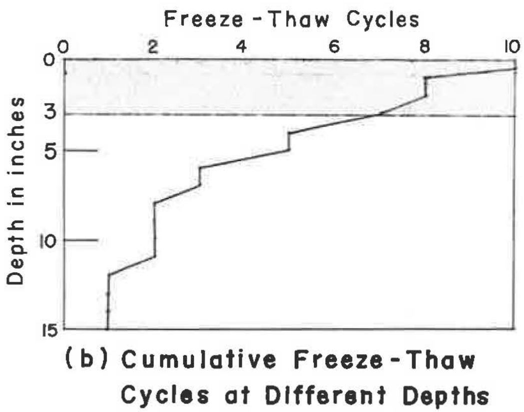


Figure 3. Freezing and thawing depths calculated from air temperature.

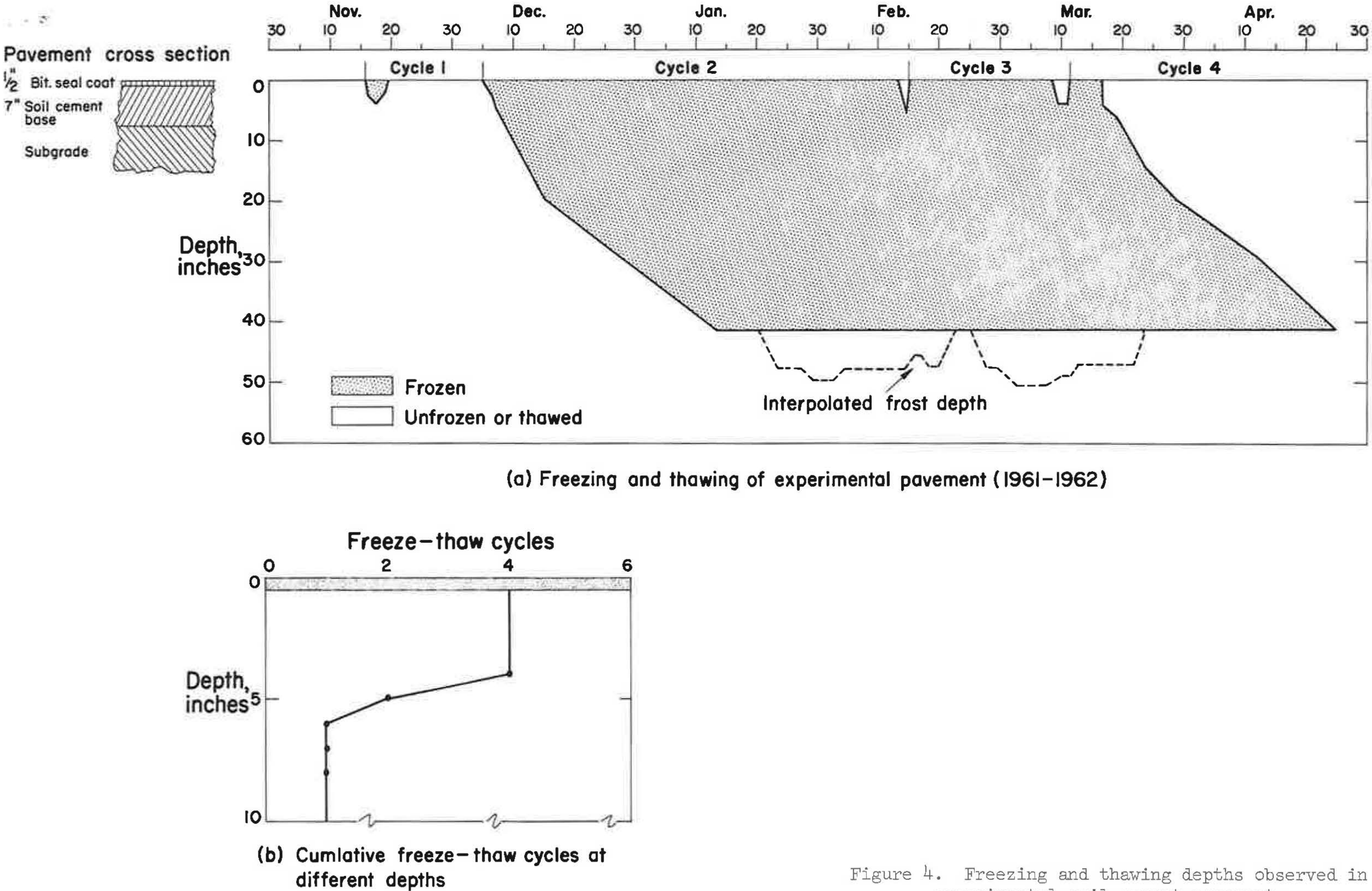


Figure 4. Freezing and thawing depths observed in experimental soil-cement pavement.

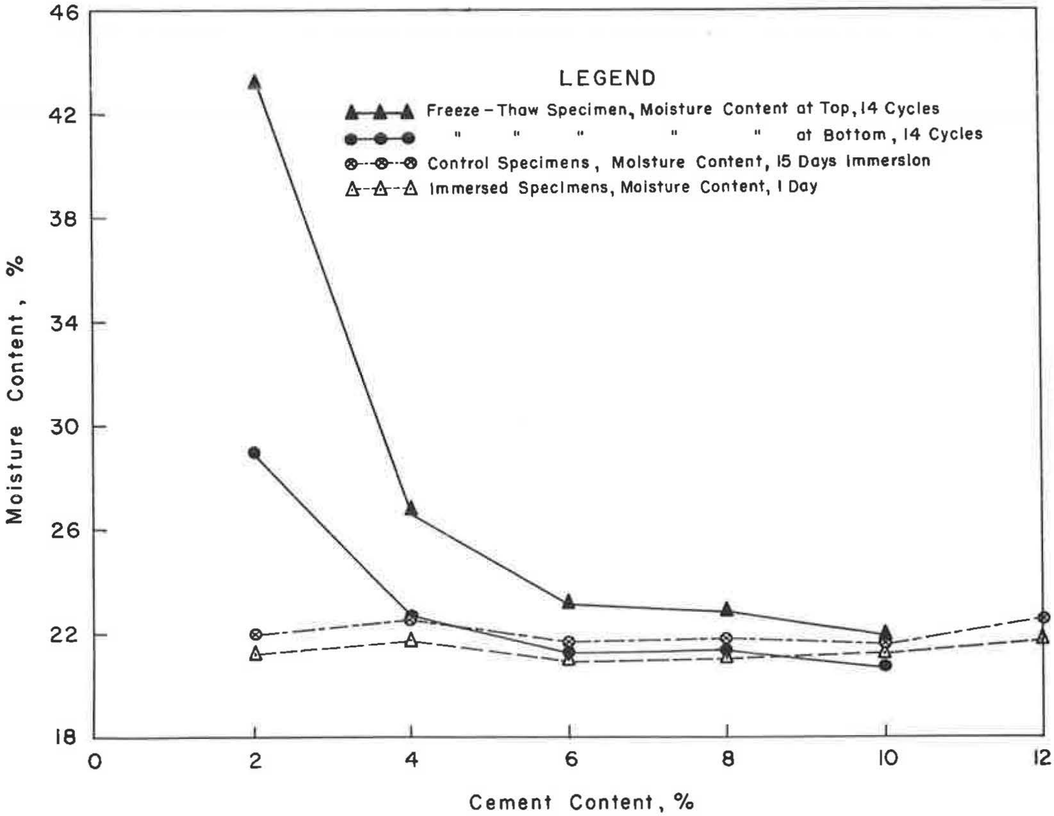


Figure 5. Effect of cement content on moisture content, and distribution of freeze-thaw test specimens.

Experimental Soil-Cement Construction

The experimental project included the placing of a soil-cement base in a 12.83-mi stretch (from Dunlap to Soldier) of Iowa primary highway 37, during August 1961. To insure uniformity in the soil-cement bases the soil was taken from a borrow area near the center of the project. The location of this area permitted central plant operation and control. The cement content required by the AASHO-ASTM standards for the available fine-grained soil was 11.0 percent (based on the dry weight of the soil). But for research purposes the cement content was varied from 7.0 to 13.0 percent in 12 experimental sections. These experimental sections are from 1,000 to 1,500 ft in length, with cement contents assigned to random test sites scattered along the entire 12.83-mi span. Cement contents of 7, 9, 11 and 13 percent were selected for the experimental soil-cement base. Some portions of the subgrade which were badly damaged were rebuilt to obtain more uniform subgrade support before the soil-cement base was laid.

Pertinent soil characteristics are given in Table 1. The borrow area was located in the Monona County thick loess area. A typical particle size distribution of the borrow soil was silt-size particles 77 percent, and 5-μ clay 21 percent with a border line classification between A-4 and A-6. The predominant clay mineral was montmorillonite. A Type 1 cement complying with section 4101 of the 1960 Iowa standard specifications was used throughout the construction. A uniform 7-in. base thickness was selected for the project.

As an aid to curing, RC-0 was applied to the compacted base within 24 hours after construction. MC-4, complying with section 4138 of the 1960 Iowa standard specifications, was used for the single bituminous coat. The ½-in. cover aggregate was crushed limestone.

TABLE 1

SOIL CHARACTERISTICS (SAMPLED DURING CONSTRUCTION) OF EXPERIMENTAL SOIL-CEMENT BASE SECTIONS

Test Section No.	Design Cement Content (%)	Properties of Natural Soil					
		Plastic Limit (%)	Plasticity Index (%)	Passing No. 200 Sieve (%)	Clay[a] <5μ (%)	Classification	
						AASHO	Textural
3	13	22	13	99	24	A-6(9)	Silty clay loam
4	7	23	11	99	18	A-6(8)	Silt
5	9	23	10	100	20	A-4(8)	Silt
6	13	23	9	99	20	A-4(8)	Silty loam
7	11	23	10	100	18	A-4(8)	Silty loam
8	7	23	12	99	24	A-6(9)	Silty clay loam
9	11	21	13	99	24	A-6(9)	Silty clay loam
10	9	22	13	100	22	A-6(9)	Silty clay loam
11	11	21	12	99	22	A-6(9)	Silty clay loam
12	7	21	10	100	22	A-4(8)	Silty clay loam
13	9	21	12	100	22	A-6(9)	Silty clay loam
14	13	20	15	99	20	A-6(10)	Silty clay loam

[a]Predominant clay mineral—montmorillonite.

Evaluation of Performance

Preliminary studies of the performance of the test sections cover a period of slightly over one year from time of construction. Pertinent field tests were conducted after the frost was out during April 1962, and in the fall of 1962 at one year. The loss of strength in the spring has been previously observed to be of the order of 60 percent (14).

Deflection as Related to Pavement Performance.—Deflections of the surface of the pavement were measured with the Benkelman beam under a moving vehicle whose axle load was 18,000 lb, the maximum allowable single-axle load in Iowa. The spring period was selected for measurement because in frost-susceptible areas flexible-type pavements are most likely to undergo distress at this time. Deflections during the spring correlate more closely with pavement performance than do those during other periods (9). In the data reported are the maximum and rebound deflections in outer and inner wheelpaths. A brief description of the procedure to obtain the deflections is presented in Hoover (10).

Results.—The relation between the maximum outer wheelpath deflection and cement content was investigated by an analysis of variance. No deflection difference was found at the 5 percent significance level between soils treated with 7 percent cement and those treated with 13 percent. Further efforts were directed towards detecting and stripping of the effects of other variables, such as California bearing ratio (CBR) of the subgrade, unconfined compressive strength of the soil-cement base, and location and lane of the Benkelman deflection data.

The CBR of the subgrade appears to influence deflections at the surface, indicated in the AASHO Test Road, where the subbase was somewhat more effective than the base in restricting deflections during both the fall and spring periods (9). The differences in the CBR of the subgrade (at the time of obtaining the deflection data) could explain part of the variation in the deflection data. In-place CBR's were measured at each location where Benkelman deflections were obtained, six for each test section. The mean value of CBR for each test section ranged from 35 to 59 percent for the weakest to the strongest subgrade.

The compressive strength of the soil-cement probably influences the flexural deflection, because the flexural strength of soil-cement is about one-fifth of the compressive strength (2). Preliminary regression analysis of the individual test sections to some extent supported this hypothesis, since two-thirds of the regression coefficients were negative, indicating that low deflection tended to be associated with high compressive strength.

TABLE 2

Factor Influencing Deflection	Regression Coefficient	t-Values, Calculated	t-Values, Tabulated[a]	Remarks
Cement, %	-2.007	3.74	1.99	Significant
Subgrade CBR	-0.011	1.91	1.99	Barely significant
Location	+0.035	4.41	1.99	Highly significant
Unconfined compressive strength	-0.001	0.12	1.99	Insignificant

[a]At 5 percent level.

The consistent increase in deflection readings from the north end of the road towards the south warranted an investigation. The finding seemed reasonable in that the clay content of the subgrade increased from north to south. Preliminary investigations showed that the increase was of significance in influencing deflection.

The effect of lane on deflection, considered a variable, later proved not very significant. Whether the sections occurred in cut or fill was not considered because the sections were randomly distributed along the road.

A multiple regression analysis was conducted to gain an insight into the factors which influence deflection under wheel loads. The deflection is determined by cement content, subgrade CBR value at 0.1 in. deflection, unconfined compressive strength of the soil-cement base, and location of stations along the test road. The outer and inner wheelpath deflections were regressed against the four variables. The same regression was repeated excluding the unconfined compressive strength, the purpose of which was to investigate the interaction between the former and the cement content to influence deflection.

The prediction equation is

$$D_{owp} = 65.7 - 2.007 (C_m) - 0.011 (C_{br}) + 0.035 (L_O) \tag{1}$$

in which

D_{owp} = outer wheelpath Benkelman beam deflection, in 0.001 in.;
C_m = cement content, percentage of dry weight of soil;
C_{br} = California bearing ratio of subgrade, lb at 0.1 in. deflection; and
L_O = location of station, in 100 ft from north to south.

The term corresponding to the unconfined compressive strength is purposely omitted because it plays little part in predicting the deflection. The regression coefficients and their test of significance are summarized in Table 2.

The regression computation for the inner wheelpath deflections led to the same general deduction in that the unconfined compressive strength was not a pertinent factor in predicting deflection.

The analysis led to some useful conclusions concerning the significance of the various parameters influencing flexural deflection:

1. The higher the cement content the lower the deflection, the rate of decrease being 2 units (0.001 in.) per percent cement content. The interval 2 ± 1 units includes the true value unless a 1-in-20 chance has occurred. (These limits are hereafter designated as 95 percent confidence limits.)
2. The CBR of the subgrade is barely significant, the higher the CBR the lower the deflection.
3. Location along the road was significant. The higher deflections were towards the south end of road, the rate of increase being 0.035 units (0.001 in.) per hundred ft of road (95 percent confidence limits 0.035 ± 0.015 units).

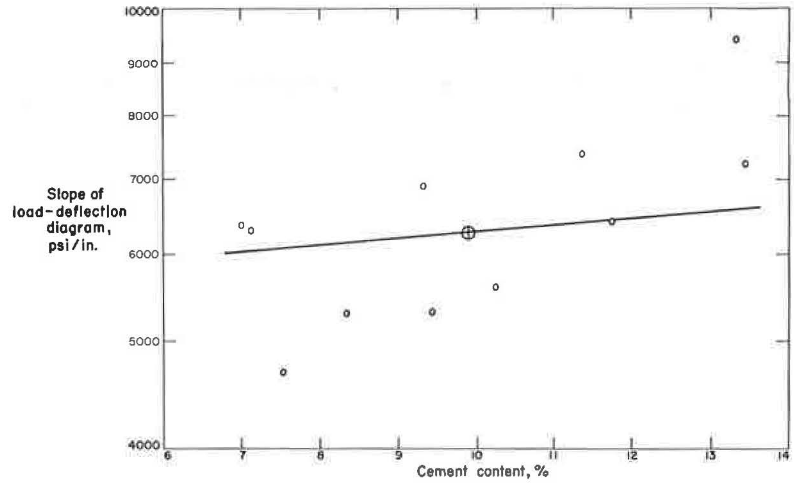


Figure 6. Effect of cement on bearing strength of soil-cement base (each point is average of six tests).

4. The unconfined compressive strength is not significant.

5. A comparison of the outer wheelpath and inner wheelpath prediction equations shows that inner wheelpath deflections are not highly influenced by the given factors, as evidenced by low t-values and small mean residual. This conclusion is in keeping with the previous finding that pavement deflection in the inner wheelpath is more uniform than in the outer wheelpath and changes only slightly with the season (6).

In the AASHO Road Test (9), the expected life of different test sections as indicated by the decline in the serviceability index to a specified value was related to spring deflections. Serviceability index, which may apply either to the present or the future, is a mathematical combination of values obtained from certain physical measurements of a large number of pavements so formulated as to predict either the present or the future ability of a pavement to serve the traffic (9). For example, a spring deflection of 0.025 in. is safe for a pavement that is expected to carry 1,000,000 18-kip axle loads without dropping the serviceability index below 2.5. Calculations on similar lines showed that the test sections built of 7 percent cement-treated base could safely carry 500,000 18-kip axle loads without dropping the serviceability index below 1.5. These predictions, however, do not take into account the gain in strength with age of soil-cement.

Bearing Capacity by Plate Bearing Tests.—Plate bearing tests were performed during April. It has been hypothesized by the Research Department of the Iowa Highway Commission that the depth of influence in the plate test is approximately $1\frac{3}{4}$ times the diameter of the plate. Thus a 4-in. diameter plate may be used to evaluate the characteristics of soil-cement base 7 in. thick with minimum influence from lower layers. The slope of the load-deflection diagram expressed in pounds per square inch per inch is reported for each location.

It was suspected that the unconfined compressive strength of the base and/or the CBR of the subgrade might influence the bearing strength as determined by plate bearing tests. Preliminary analysis showed that these factors did not have any significant influence on the slope of the load deflection diagram. However, the bearing strength shows a slight increase with increase in cement content (Fig. 6). The presence of closely spaced cracks in soil-cement of 7 percent admixture might be one of the reasons contributing to its larger deflection.

Surface Crack Studies.—Cracking is an element of structural deterioration that detracts from serviceability and performance of flexible pavement. Cracks do not in themselves have much effect on the ability of the pavement to serve traffic, provided

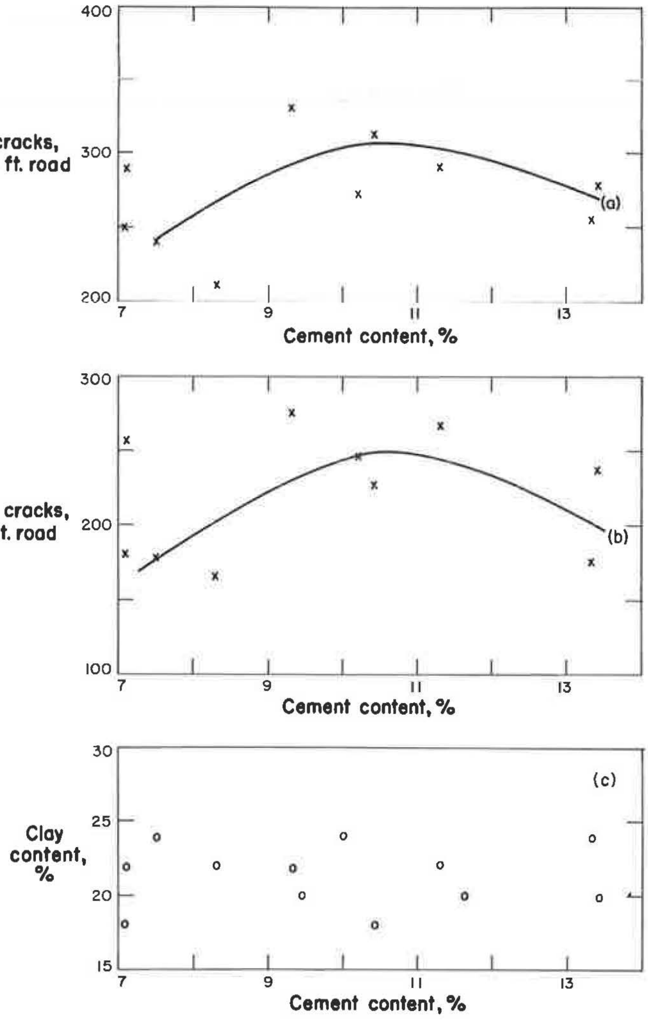


Figure 7. Relation between extent of (a) transverse cracks and cement content, (b) longitudinal cracks and cement content, and (c) clay content of twelve different test sections and corresponding cement contents.

they are intact. However, weakening of the pavement structure is likely because of infiltration of surface water.

More cracks appeared during periods of cold weather, at which time a crack survey was conducted. The results obtained, expressed in lineal feet per 100 feet of pavement, are plotted against cement content (Figure 7 a and b). The extent of cracking seems to be optimum for a soil-cement whose cement content is around 10.5 percent.

Before an explanation for this maximum cracking is offered, the various factors causing shrinkage and cracking should be analyzed. Shrinkage cracks are inherent in soil-cement. Recent research in shrinkage of soil-cement by H. Nakayama and R. L. Handy indicates that this phenomenon is primarily influenced by the amount of 5-μ clay in the soil. Cracks in soil-cement also could be flexural failures. The reasoning is essentially as follows: the flexural strength corresponding to an unconfined compressive strength of 300 psi is of the order of 60 psi (2). Using the Westergaard method of rigid pavement design, flexural stresses of the order of about 100 psi should be devel-

Figure 8. Crack pattern of 4-in. diameter road cores with cement contents of 13 and 7 percent after one year.

oped under a wheelload of 18,000 lb, which is far above the cracking stresses. Had shrinkage cracks not occurred, flexural cracks would have, causing the soil-cement to act as a flexible material.

The clay content of the soil-aggregate that went into the soil-cement remained almost the same for all sections (Fig. 7c).

As the cement content is increased, the flexural strength is also increased, the beam action is more pronounced as a result, and flexural cracks occur at longer spacings. The adverse effect of high cement content on shrinkage is overrun by the higher flexural strength.

At lower cement contents the cracks were closely spaced. Being very fine, they are likely to be overlooked and often they were not visible on the pavement surface. Numerous concealed cracks revealed during coring operations supported this hypothesis.

Several cores drilled after one year were examined for defects. These were mainly vertical cracks whose width and depth were noted and the crack plane of each was checked for direction and interlocking characteristics. The results indicated that cracking is inevitable and to some extent is required of soil-cement construction. Closely spaced shrinkage cracks were observed in all test sections. Of the 66 cores examined, 39 exhibited essentially vertical cracks (Fig. 8). These data support the finding of Mitchell (13).

Unconfined Compressive Strength of Road Cores.—Summarizing the analysis of the pavement performance, it may be concluded that test sections built of 7 percent cement-treated soil were as structurally sound as the sections built of high quality soil-cement. However, it may be pointed out that future structural deterioration of the test pavement is unlikely because soil-cement gains strength with age, and the first spring thawing period is the most critical time. This can be verified by comparing the core strengths at the age of eight days with those at 245 days during spring thaw, and at the age of one year, in August 1962. The core strengths are graphed against the log of the age of soil-cement (Fig. 9), because strength of soil-cement correlates best with time of curing in a semilogarithmic manner (3). The relatively flat slope of strength-age relationship during the winter is an indication of the retarding effects of freezing temperatures.

CRITERIA FOR DESIGN OF SOIL-CEMENT

Most highway agencies base the mix design of soil-cement on the standard durability tests, which have been successfully used for many years. A soil-cement base must remain stable to resist effects of weathering and traffic. The durability tests primarily measure deterioration or weight loss of specimens by physical weathering, and it is implied that the soil-cement which meets the durability requirements satisfies strength requirements as well. However, it would be desirable if both requirements could be met in one test procedure. This requirement may be simplified in that failure of a soil-cement base by deformation is most likely a shear failure; therefore, a safe minimum allowable unconfined compressive strength after adequately severe simulated weathering treatments may be considered as a valid criterion of strength and durability (4). Simplicity and reliability of test procedures are other reasons for favoring unconfined compressive strength criteria.

The performance of the 7 percent soil-cement mixture in the field test sections is satisfactory. However, the question arises as to the successful performance of a soil-cement mixture of lower cement content. Though a 5 percent soil-cement mixture was

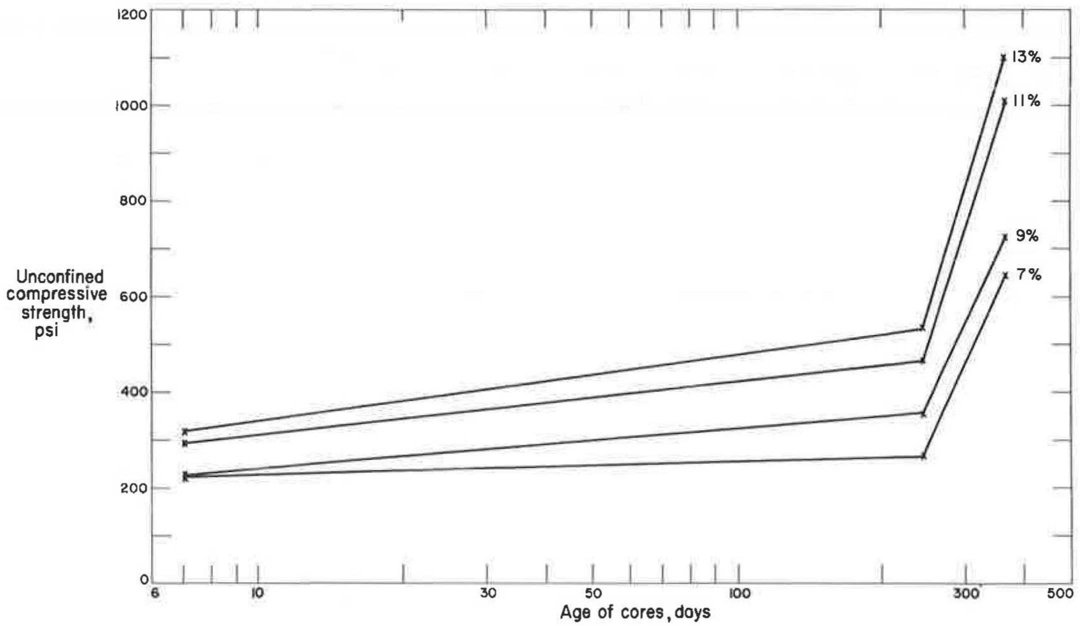


Figure 9. Effect of age on unconfined compressive strength of Proctor-size road cores after immersion in distilled water for 24 hours before test.

not a part of the experiments in the field the indications are that it may not perform satisfactorily under field freezing and thawing conditions. Figure 10 shows two test specimens which were subjected to Iowa freeze-thaw tests. That the 5 percent soil-cement mixture did not survive the freeze-thaw test in the laboratory indicated its inadequate strength gaining capability. Therefore, the adequate and economical cement content to use would be the lowest percent admixture that survived in the field and laboratory. The properties of a soil-cement mixture with 7 percent of cement will be the tentative criteria for mix design with fine-grained soils. The results given are for soil samples obtained from the borrow area during the time of construction of the experimental base. Details regarding molding, curing and testing of the 2-in. diameter by 2-in. high specimens have been given (4).

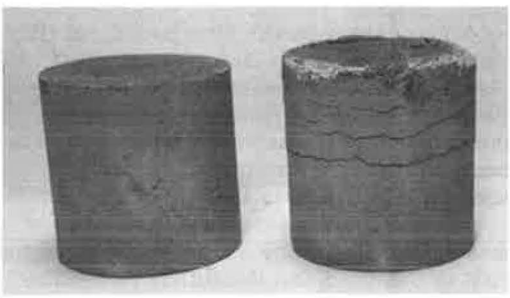


Figure 10. Freeze-thaw deterioration of 2-in. diameter by 2-in. high test specimens with cement contents of 7 and 5 percent after 10 cycles of freezing and thawing.

Results.—Based on the analysis and acceptable performance of the 7 percent test section, the following criteria may be used as the basis for selecting minimum cement contents for durable soil-cement. Mixes are prepared in the laboratory and molded into 2-in. diameter by 2-in. high test specimens.

1. The minimum unconfined compressive strength shall be 453 psi when tested after 7 days' curing in a humid room followed by 24 hours' immersion in distilled water, 95 percent confidence limits from 431 to 475 psi.

2. The minimum unconfined compressive strength after being subjected to the Iowa freeze-thaw test (Appendix A) is 500 psi, 95 percent confidence limits from 472 to 528 psi.

Seven-day-cured and one-day-immersed strengths may be reliably used for preliminary studies since they correlate with the corresponding freeze-thaw results; the correlation coefficient is 0.869, based on analysis of 40 pairs of strength results with a wide range of cement contents. Strength after freeze-thaw may be predicted by multiplying the 7-day strengths by 1.221 ± 0.342 (95 percent confidence limits on the prediction coefficient). A statistical t-test was conducted on the regression coefficient to test the approach to unity; the ratio may be considered 1.0, subjected to a chance variation of 1-in-20.

Another criterion is a measure of the relative strength gaining capability of soil-cement during adverse conditions of freezing and thawing. This measure is obtained by dividing the strength at the end of the test by the continually immersed specimen at room temperature. The indices of resistance of soil-cement mixtures are plotted against the corresponding cement contents (Fig. 11). Also shown is the ratio of the unconfined compressive strengths of specimens buried adjacent to the test road and similar specimens stored in the humid room for a 1-yr period. These specimens were molded from the soil-cement mixture mixed in the pugmill during construction of the test sections.

Comparison of a and b (Fig. 11) shows either that the laboratory freeze-thaw test is more severe than what actually occurred in the field, or that transient field temperature and moisture conditions provide better curing than the carefully controlled conditions of the laboratory moist-cure room. The latter explanation appears more acceptable, since field-cured strengths of hand-molded specimens with 11 and 13 percent cement actually exceeded those obtained in the laboratory.

The index of resistance drops below 80 percent for mixtures of less than 7 percent cement. Therefore, the third criterion tentatively reads as follows:

3. Laboratory test specimens shall give a minimum index of resistance to freezing of 80 percent after being subjected to Iowa freeze-thaw test.

Strength Criterion by Correlation.—A correlation for the 8-day unconfined compressive strength with the ASTM weight loss has been established (correlation coefficient 0.646), the test specimens being road cores from the experimental soil-cement base (Fig. 12a). Because these results cover a wide range of cements feasible for soil-cement, and because the properties obey a linear relationship in the range of cement

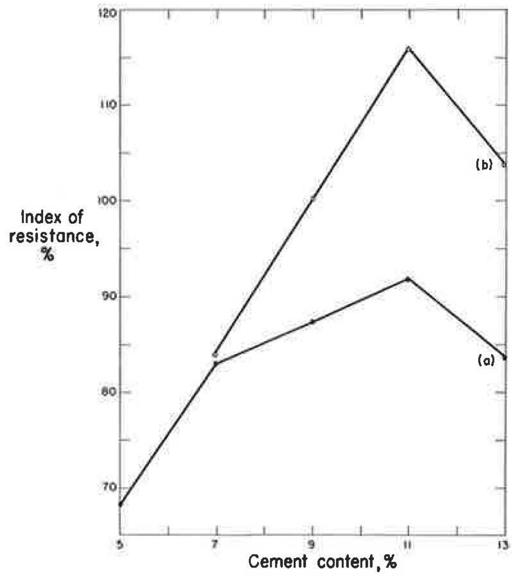


Figure 11. Relationship between (a) index of resistance to freezing and cement content and (b) index of resistance to freezing of field weathered test specimens and cement content (2-in. diameter by 2-in. high specimens).

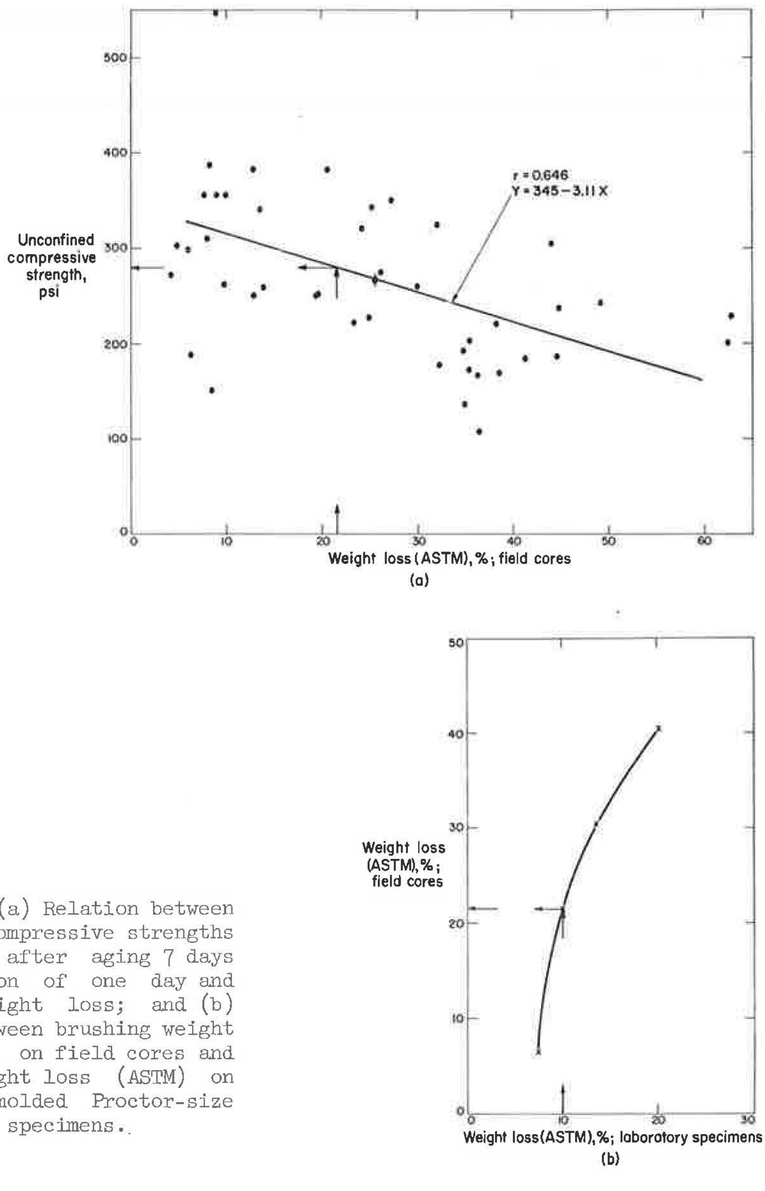


Figure 12. (a) Relation between unconfined compressive strengths of specimens after aging 7 days and immersion of one day and brushing weight loss; and (b) relation between brushing weight loss (ASTM) on field cores and brushing weight loss (ASTM) on laboratory molded Proctor-size specimens.

contents, the prediction of unconfined compressive strength from weight loss is permissible. The expected unconfined compressive strength is

$$Y = 345.3 - 3.108 X \tag{2}$$

in which

Y = unconfined compressive strength (psi) of road cores at 8 days following 24-hr immersion, and
X = ASTM brushing weight loss.

The permissible weight loss of this soil is 10 percent (ASTM Designation D560-57) to produce durable soil-cement. Corresponding weight loss for road cores of similar soil-cement as that of laboratory specimens would be predicted from a plot of weight loss on laboratory specimens against the laboratory results on field cores. The predicted weight loss is 21.5 percent (Fig. 12b). Knowing the weight loss, the expression can be used to predict the expected compressive strength.

When X = 21.5, the population mean of the unconfined compressive strength is estimated at 278 psi with 95 percent confidence limits from 251 to 305 psi. In other words if a field strength of 250 to 300 psi is guaranteed, the soil-cement would necessarily meet the requirements of durability. In the United Kingdom, a minimum compressive strength of 250 psi at 7 days has been used successfully to design soil-cement for bases of lightly trafficked roads, but for heavily trafficked roads, a minimum strength of the order of 400 psi at 7 days is probably required (4). An immersed strength of about 275 psi after 7 days' curing has been suggested by the soil-cement bureau of the Portland Cement Association as tentative strength value for friable loess.

It has been found that the construction methods used in the soil-cement base construction result in a field strength of about 55 to 65 percent of the laboratory results. Experience in Great Britain showed a value of 60 percent was probably satisfactory (2, 13).

The cement content is therefore that necessary to give a laboratory compressive strength equal to 278/0.6 = 463 psi. Thus, the 7-day immersed compressive strength would be 463 ± 45 psi. The statement is correct except for the diverse kind of sample that occurs about once in 20 trials.

CONCLUSIONS

An Iowa freeze-thaw test, modeled after the modified British freeze-thaw test but designed to simulate more closely winter field conditions in Iowa, has been developed (Appendix A). Essentially the Iowa freeze-thaw test method covers the determination of the change in the unconfined compressive strength of specimens of stabilized fine-grained soil when subjected to cycles of freezing and thawing under conditions summarized as follows:

1. A freezer temperature (20 F) equivalent to the daily average minimum air temperature in Iowa during winter months.
2. A meaningful temperature (35 F) inside the vacuum flask at the bottom of the specimen simulating the soil temperature during winter months.
3. The number of freeze-thaw cycles (10) is obtained from a plot of the average air temperature, in that freezing of pavements is considered to occur when the air temperature falls below 32 F, and thawing when the temperature increases above 32 F.
4. For simplicity of procedure, the specimens are to be thawed at room temperature, 77 F.
5. The important criteria for the evaluation of the Iowa freeze-thaw test are the unconfined compressive strength and index of resistance to freezing. Heave upon freezing and the moisture content and distribution after freeze-thaw tests offer good promise as criteria.

To establish design criteria, several sections of Iowa primary highway 37 were built with soil-cement mixtures with varying cement contents. The central plant construction insured uniform soil characteristics. Cement contents ranged from 7 to 13 percent whereas the ASTM Method D560-57, warranted the use of 11 percent cement.

The experimental test sections were evaluated on the basis of the Benkelman beam deflection, plate bearing tests, crack survey, and the unconfined compressive strengths of road cores. Based on the acceptable performance of the 7 percent test section, the laboratory test results of 2-in. diameter by 2-in. high test specimens of the soil-cement are proposed as tentative criteria for the design of soil-cement:

1. The minimum unconfined compressive strength shall be 433 ± 23 psi when tested after 7 days' curing followed by 24 hours' immersion in distilled water.
2. The minimum unconfined compressive strength after being subjected to the Iowa freeze-thaw test shall be 459 ± 41 psi.
3. Laboratory test specimens shall give a minimum index of resistance to freezing of 80 percent after being subjected to the test.

In view of the correlation (r = 0.646) between the unconfined compressive strength and the ASTM brushing weight loss on road cores, the validity of the 7-day strength criterion is verified.

ACKNOWLEDGMENTS

This report was part of the research under Project 451-S of the Engineering Experiment Station, Iowa State University, under the sponsorship of the Iowa Highway Research Board, Iowa State Highway Commission.

The authors extend appreciation to R. L. Handy, H. T. David, J. M. Hoover, and T. Demirel.

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Appendix A

IOWA FREEZE-THAW TEST

Scope

1. This method covers the determination of the change in the unconfined compressive strength of 2-in. high by 2-in. diameter specimens of stabilized fine-grained soil when subjected to cycles of freezing and thawing under specified conditions which simulate winter climate in Iowa. The Iowa freeze-thaw test is a modification of the British (B. S. 1924: 1957) freeze-thaw test.

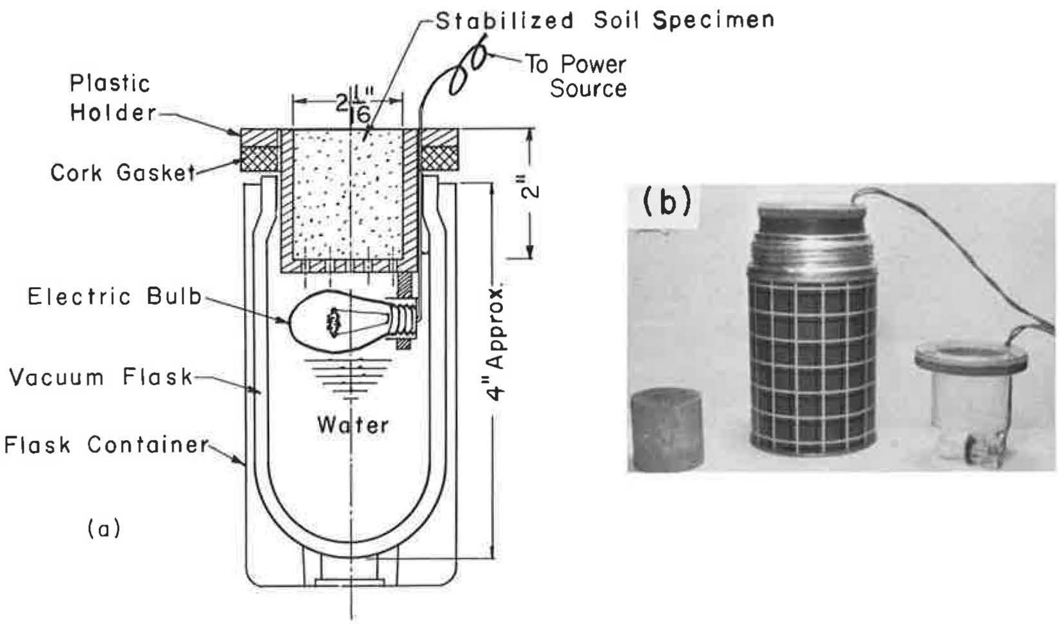

Figure 13. (a) Freeze-thaw test apparatus; (b) vacuum flask, stabilized soil specimen and specimen holder.

Apparatus

2. The apparatus required (Fig. 13) is as follows:

   a. A commercial vacuum flask having a neck with an internal diameter of approximately 2.5 in. and an internal depth of at least 4 in.
   b. A specimen holder of low thermal conductivity and resistant to deformation under the test conditions, and capable of supporting a stabilized specimen 2 in. high and 2 in. in diameter within the vacuum flask, so that the upper flat surface of the specimen is flush with the top of the flask (Fig. 13). The base of the carrier is perforated to permit free access of water to the underside of the specimen. A heating element with an output of about 0.5 watt is attached to the bottom of the specimen holder (a 6-watt bulb connected through a variable transformer using a 110 ac supply is found to be very convenient).
   c. A refrigerated space maintained at a temperature of 20 ± 2 F which is large enough to contain the vacuum flask with its enclosed specimen. A thermometer is installed inside the refrigerated space.
   d. A supply of asphalt or resin-base paint.
   e. A supply of self-adhering membrane (the commercial product Saran Wrap is satisfactory).
   f. About 100 ml of distilled water, cooled to 35 ± 2 F.
   g. A height gage equipped with an Ames dial having an accuracy of 0.001 in.

Preparation of Specimens for Test

3. For each determination two identical specimens 2 ± 0.05 in. high and 2 in. in diameter are prepared as has been described (4). (If greater accuracy is desired, 6 or 8 identical specimens may be prepared for each determination).

Test Procedure

4. a. After the desired curing period of 7 days' moist curing, any covering material on the specimens is removed and both specimens are weighed. If moist

curing either specimen has lost more than two grams in weight during storage at a temperature of 70 ± 1 F and a relative humidity of at least 95 percent, both specimens are discarded.

b. After weighing, a coating of asphalt or resin-base paint about 1 mm thick, is applied to the top surfaces of both specimens and allowed to dry. The specimens shall then be immersed in distilled water at 77 ± 4 F.

c. After immersion for 24 hours, the specimens are removed from the water and dried with blotting paper. The height and weight of the specimens are measured. One specimen is subjected to freezing and thawing and the other to immersion. A collar, 1.5 in. deep, of a self-adhering membrane is placed around the top of the freeze-thaw specimen.

d. Sufficient water at a temperature of 35 deg shall be poured into the vacuum flask so that when the specimen dealt with in c above is inserted in the holder and the latter placed in the flask, the bottom ¼ in. of the specimen is immersed in water. The height of the specimen while inside the holder shall be measured using the Ames dial. The vacuum flask and its contents is placed in the refrigerated space maintained at 20 ± 2 F. As the water inside the vacuum flask cools down (after about one hour) it is heated up and brought to the required temperature of 35 ± 2 F. The vacuum flask is kept in the refrigerated space for 16 hours.

e. The flask and contents are removed and the height of the specimen while inside the holder measured. The specimen is thawed for 8 hours at a temperature of 77 ± 4 F. If, after thawing, the level of the water inside the vacuum flask has dropped so that it is no longer in contact with the base of the specimen, water at 35 F is added to restore the level.

f. The procedure described in d and e above constitutes one cycle of freezing and thawing. Testing shall continue until the specimen has been subjected to 10 such cycles.

g. At the conclusion of the freezing and thawing cycles, the thawed specimen is removed from the holder and, together with the second specimen control which has been stored in water during the entire period, allowed to drain for 15 minutes. The heights and weights of both specimens are determined.

h. The unconfined compressive strengths of the two specimens are then determined. Each specimen is placed centrally on the lower plate of the compression testing machine, and the load is applied so that the rate of deformation is uniform and approximately 0.10 in./min. The maximum load in pounds exerted by the testing machine is noted and recorded (p_f for the freeze-thaw specimen and p_c for the control specimen).

i. The moisture contents of representative samples of fragments taken from the specimens are determined.

Calculations

5. a. The unconfined compressive strengths (p_f and p_c) of the two specimens are calculated from the formula:

$$p = 0.318 \text{ P (psi)}$$

where P = the maximum load recorded in pounds.

b. The index of resistance to the effect of freezing (R_f) is calculated from the formula:

$$R_f = \frac{100\, p_f}{p_c} \text{ (\%)}.$$

Reporting of Results

6. a. The important criteria used to evaluate the stabilized soils are p_c, p_f, and R_f, which are reported.

b. The report may include the "heave" expressed as a percentage of the initial height and the final moisture content and its distribution.

c. Also the details of composition of the stabilized soil mixture and the dry density at the time of molding are reported.