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Emulsion Mix Design, Stabilization, and Compaction

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Emulsion Mix Design Methods: An Overview

H. FRED WALLER, JR.

A general overview is presented of several methods of emulsion mix design currently being used either experimentally or on an operating basis. Most of the methods make use of either Hveem or Marshall equipment for molding and testing specimens of the mixture. In most cases, modifications have been made to standard procedures to accommodate the special requirements of emulsified-asphalt mixes. Although laboratory procedures may differ in each case, the test methods must generally address the following problems: (a) the amount of mixing water required as an aid to proper coating and workability, (b) the type and grade of emulsified asphalt to be used, (c) the amount of emulsified asphalt required for optimum results, (d) the curing rate of specific mixes, (e) the water sensitivity of the mixture, (f) some measure of strength or load-carrying capability, (g) the tendency of the emulsified asphalt to drain off the aggregate (in open-graded mixes) before a sufficient thickness has adhered to the aggregate surface, (h) the optimum mixing time to ensure proper coating but not to the extent that the emulsified asphalt is stripped from the aggregate, and (i) a laboratory compactive effort that will produce a density comparable to that obtained in the field. Some of the problems associated with emulsified-asphalt mix design are examined, and the known test procedures used to achieve the objectives of an acceptable mix-design system are summarized.

In recent years, the use of emulsified-asphalt mixes in road construction has gained wide acceptance. Engineers have learned that mixes of this type can be designed and constructed so as to offer performance characteristics comparable to those of hot plant mixes. In addition to the obvious energy savings, the use of emulsified asphalt provides significant economic advantages because it permits a wide latitude in the selection of aggregate gradations and quality standards (1).

Because of environmental and energy considerations, the U.S. Environmental Protection Agency, the Federal Energy Administration, and the Federal Highway Administration (FHWA) have encouraged greater use of emulsified asphalt. Such federal action has been partly responsible for a gradual increase in the use of emulsion. In 1978, for the first year in history, the consumption of emulsions exceeded that of cutback asphalt. Of the 32 million Mg (35 million tons) of asphalt used in 1978, about 8.4 percent was asphalt emulsion and 7.3 percent was cutback asphalt.

Much of the early history of emulsion-aggregate mixes involved the in situ stabilization of sands and local fine aggregates. But the development of mobile, highly efficient mixing plants for emulsified-asphalt mixes (EAMs) now permits a much broader range of mixture types. Furthermore, these plants can be set up in remote areas where the cost of hot plant mixes would be economically prohibitive. Experience with EAMs in logging operations conducted by the U.S. Forest Service has shown that these mixes can support loads of approximately 889 kN (200 000 lbf) without undue distress (2).

In 1978, the Asphalt Institute, under contract to the Asphalt Emulsion Manufacturers Association and FHWA, produced a publication entitled A Basic Asphalt Emulsion Manual (1). As an adjunct to the main objective of this effort, data on various EAM design methods were also assembled. Eleven different methods were discovered during the collection of material for the manual. FHWA published all 11 procedures as Volume 2 of their version of the manual (3). The Asphalt Institute included only 2 procedures as part of its basic manual, one based on the Hveem procedure and the other on the Marshall procedure (1).

In the latter part of 1979, the Asphalt Institute began two comprehensive studies of EAM design procedures. The first is an in-house project funded by the Asphalt Institute. The following specific tasks are included:

1. Investigate the use of a range of emulsion contents for preparing trial mix specimens based on residual asphalt in the emulsion for various gradings.
2. Investigate the use of existing coating tests or modifications of these tests for selecting the type of emulsified asphalt and for determining water content for mixing.
3. Investigate determinations of air-void contents and voids in mineral aggregate of compacted EAMs and possible use of these properties as criteria for mix design.
4. Investigate retained Marshall stability of EAMs after various conditions of exposure to water or moisture vapor.
5. Prepare a draft of the Marshall mix-design procedures for EAMs.

The primary objective of the second study, which is being funded by the National Cooperative Highway Research Program, is to verify and/or modify the EAM design methods of the Asphalt Institute and the University of Illinois. The applicability of these mix-design procedures and criteria to prediction of the field performance of base- and surface-course mixtures will be determined. The base- and surface-course mixtures studied will involve both slow- and medium-setting emulsions as well as various types, gradings, and qualities of aggregate. The evaluation will also consider the relations between the properties of the mixtures as determined in the laboratory and the rate of attainment of these properties in the field under different environmental conditions.

Reports on both studies will be published when the research is completed.

GENERAL CONSIDERATIONS OF EAM DESIGN

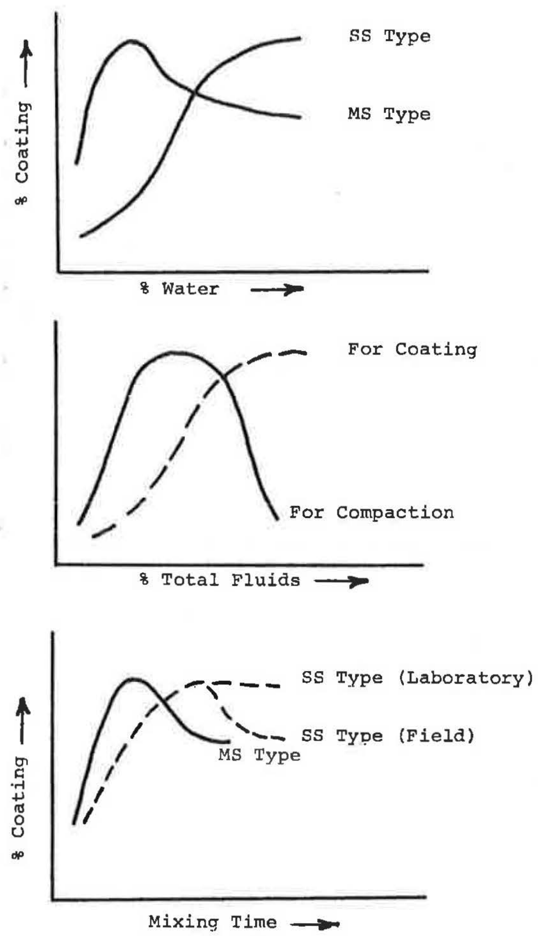
The development of a standardized procedure for the design of mixtures containing emulsified asphalt and aggregate presents a significant challenge to the highway industry. Although considerable research has been done in this area, unanimous agreement has not been reached and much research remains to be done (4).

Standard design methods for hot plant mixes are readily available, and technologists are in good agreement as to the validity of test results obtained and their specific application. The design of emulsion mixes is considerably more complex because of the difficulty of duplicating the field curing of EAMs in the laboratory. In mix-design procedures for hot plant mixes, ultimate stability and related mix properties are reached at or near the time when the test specimens are formed. In emulsion mixes, ultimate stability and related properties are not reached until virtually all water in the mixture has evaporated. Under field conditions, this evaporation may require several months, even as much as two years.

To measure the test properties of EAMs, it is necessary to devise some type of laboratory curing procedure. The degree and rate of curing should bear a known relationship to the curing of the mixture in the field. Laboratory methods used to remove the water and the rate of water removal can have a significant effect on the test values obtained. Too-rapid evaporation of water by oven drying may be unrealistic and diminish the value of the test properties (5).

Some of the test procedures involve air drying, whereas others involve oven drying (curing). In some cases, mixtures are left in the forming mold for curing so that air does not penetrate the interior of the specimen. Whether the mixture is open graded or dense graded will also affect the specific approach used and the type of data needed. Thus, the interpretation of laboratory test results and their correlation with field performance is not always clear and direct.

Figure 1. EAM characteristics.



Some of the general aspects of the coating and mixing of EAMs are shown in Figure 1. The figure is conceptual in nature; i.e., it is not based on specific laboratory test data. It can be seen that the percentage of water, percentage of total fluids, and mixing time are critical considerations with respect to coating and compaction (6).

Irrespective of the approach or the type of test equipment used in emulsion mix design, certain basic questions must be answered by the procedure:

1. What type and grade of asphalt emulsion will be most compatible with the aggregate in question?
2. What aggregate type and gradation will best fit the traffic and environmental requirements?
3. How much mixing water must be added to achieve the desired degree of coating and workability in the mixture?
4. What percentage of emulsion must be used to achieve optimum mixing conditions and provide a sufficient coating of residual asphalt on the aggregate particles?
5. What strength level or load-carrying capacity will the mixture produce under optimum conditions?
6. Will the mixture be water sensitive (subject to stripping) if adverse moisture conditions are present?
7. Will the addition of a small quantity of mineral filler or portland cement be necessary to develop early strength or to prevent runoff of the emulsion, particularly with open-graded mixes?
8. How should the mixture be "laboratory cured" in order to duplicate field curing conditions?
9. What relationship will the laboratory test data have to the field performance of the mixture? Can probable success be predicted with a reasonable degree of assurance, based on the test data?

Standards of the American Society for Testing and Materials describe 16 different types of emulsified asphalt. Each is designed for a specific function, although some grades can be used for multiple purposes. For emulsion-aggregate mixes, it is probable that a medium-setting emulsion would be selected for an open-graded mix and either a medium- or slow-setting emulsion for dense-graded mixes. It is unlikely that a rapid-setting emulsion would be used because it tends to break before the aggregate is properly coated and the mixture spread. The choice between anionic and cationic emulsion depends largely on the type of aggregate involved. The specific grade is determined by laboratory tests that include a visual inspection of the coating achieved by one or more different grades of emulsion. The aggregate gradation has a significant effect on the selection of the grade of emulsion to be used.

Emulsion-aggregate mixes offer a wider choice of aggregate gradations than do hot plant mixes with asphalt cement. Although the use of "dirty" aggregate is not recommended, certain types of emulsified asphalts are able to provide a reasonably good coating on aggregates that would not be used in hot-plant-mix operations. Emulsion used for this purpose normally contains a small percentage of solvent. As a general rule, an aggregate could be considered for EAMs if (a) it has less than 20 percent passing the 0.074-mm (No. 200) sieve, (b) it has a sand equivalent of 25 or more, and (c) it has a plasticity index that, multiplied by the percentage passing the 0.074-mm sieve, does not exceed 72. Very dense-graded aggregates may present a special problem in that a sufficient amount of fluids (emulsion plus mixing water) cannot be incorporated because of limited void space (6).

In virtually all EAMs, it is necessary that some percentage of mixing water be added to the aggregate to facilitate coating and workability of the mixture. This is largely a trial-and-error proposition. The amount of mixing water must be sufficient to aid in good distribution of the emulsion over the aggregate surface, yet water should not be used to the extent that the emulsion will drain from the aggregate particles (7). Complete asphalt coating is not necessarily required in order to produce a satisfactory mix. In a condition known as "graybacks", part of the aggregate surface may appear to be uncoated and yet the mix will be satisfactory in the field.

The desired percentage of residual asphalt is the major factor in determining the amount of emulsion to use. It must be remembered that the emulsion contains about one-third water, which is lost in the curing process. The required amount of residual asphalt will be about the same as the amount of asphalt cement required for a hot plant mix. Factors such as void content, voids filled with asphalt, degree of aggregate coating, stability, and stiffness modulus must be taken into account in arriving at the optimum emulsion content.

The asphalt mixture must possess sufficient stability or resistance to deformation to support anticipated traffic loads without cracking, rutting, or distorting. In many EAMs, stability continues to increase over time as water evaporates. The mix-design procedure must be able to measure initial and long-range stability. Furthermore, a correlation must be made between laboratory design and field performance requirements.

The test procedure must provide some indication of the water sensitivity of the EAM. If there is an indication of stripping or hydrophilic tendencies, it may be desirable to incorporate an antistrip additive to the emulsion. An unrealistic laboratory curing procedure could greatly distort the measurement of stripping.

It has been found that the addition of a small quantity of mineral filler or portland cement may improve early strength and make the mixture more resistant to freeze-thaw cycles. The addition of cement may also aid in preventing excessive runoff in certain types of EAMs. As a general rule, the maximum amount of cement or filler should not exceed a ratio of about one part to five parts of emulsion by weight (7).

SUMMARY OF KNOWN MIX-DESIGN METHODS

For any test measurement to have full value, it must bear a known relationship to the same value of the mixture in the field. This means that the mixing, compaction, and curing methods used in the laboratory must provide test specimens that are very similar to the same mixture in the field and there must be a predictable relationship between the properties in the laboratory and those in the field. Several different approaches are currently being used to provide some of these answers.

The following summaries of emulsion-mix-design procedures are based on Volume 2 of A Basic Asphalt Emulsion Manual (3) as published by FHWA. Complete details of each procedure are provided in that publication.

Asphalt Institute Method

The Asphalt Institute method covers the selection, proportioning, and testing of aggregates, additives, and emulsified asphalt for mixes to be used in pavement construction. The procedure is a combination of test methods of the California Division of Highways as well as procedures developed within the Asphalt Institute. The major steps involved in the Asphalt Institute procedure are shown in Figure 2. The centrifuge kerosene equivalent (CKE) test is used in estimating the emulsified-asphalt contents for trial mixes of aggregates (other than open graded).

The percentage of emulsified asphalt by weight will generally be in the range of 5-10 percent for dense-graded mixes, 4.5-8 percent for fine-aggregate mixes, 4.5-6.5 percent for open-graded coarse mixes, 5.0-7.0 percent for open-graded medium mixes, and 6.0-8.0 percent for open-graded fine mixes. Mixing by either spoon and bowl or mechanical means is done to determine the coating, workability, and runoff (open-graded mixes only) of the trial mixture. The optimum fluid content (mixing water plus emulsified asphalt) for compaction and test-specimen fabrication is determined by a light kneading compaction followed by application of a double-plunger static load.

The strength of emulsified-asphalt mixes is measured by running a final modulus at a temperature of 23° ± 1.7°C (73° ± 3°F) after a total of three days of cure in the mold plus four days of vacuum desiccation. These data are used in conjunction with certain project variables (traffic, regional temperature, and curing conditions) and other mix properties (volume percentage of asphalt residue and air voids) in determining the pavement thickness requirements.

The strength of base and temporary surface mixes (other than open graded) is evaluated before and after vacuum saturation. Base and temporary surface mixes are tested at 23° ± 3°C (73° ± 5°F) for resistance (R-value) and cohesion (C-value). Surface mixes are tested at 60° ± 3°C (140° ± 5°F) for their stabilometer S-value and cohesiometer C-value.

If rain is a possibility on a project within a short period after laydown, open-graded mixes are evaluated for damage by surface water. Under favorable curing conditions, after 24 h, damage from washoff as a result of rainfall is generally not a problem. Non-open-graded mixes that are used in the base course or as a temporary wearing surface are evaluated for early strength and fully cured strength after vacuum saturation.

The items of test data recommended for inclusion in a report on EAM design are given in Table 1.

U.S. Forest Service Method

Dense-Graded Mix

The method used by the U.S. Forest Service to determine the proportions of dense-graded aggregate, emulsified asphalt, and water that will yield a workable paving mixture uses the CKE test, mixing tests, and split-tension tests on specimens compacted with a kneading compactor. The method, including preparation and preliminary tests, normally requires about 48 person-h over a period of 13-15 workdays.

The amount of added water for mixing is determined by adding to the first 500-g aggregate sample the minimum amount of water required to uniformly darken the aggregate. The aggregate is stirred until the water is evenly distributed. The emulsified asphalt is then added and mixed by hand for 30 ± 5 s. The workability of the mixture is then recorded as good, fair, or poor. After curing overnight, the asphalt coating is recorded as thin, moderate, or heavy, and an estimate is made of the percentage of aggregate area coated. Several 500-g batches are made in this way by using a constant emulsion content and varying the water content by 1 percent for each batch. These mixing tests are continued until the minimum moisture content at which the mix has at least fair workability, 90 percent coated area, and a moderately heavy to heavy coating is found. A total of eight specimens are prepared with the optimum fluid content. The specimens are cured by drying in an oven for 24 h at 49° ± 1°C (120° ± 1.8°F).

Half of the specimens are tested in a dry condition and half after 24-h water immersion by using the split tension test. The test data are plotted on four graphs: (a) dry tensile strength versus emulsion content, (b) wet tensile strength versus emulsion content, (c) index of retained strength versus emulsion content, and (d) dry density versus emulsion content.

The report of the laboratory test results includes the following information: gradation, aggregate specific gravity, emulsion type, percentage emulsion content by dry weight of aggregate, lower-limit percentage moisture content, upper-limit percentage moisture content, maximum dry density, and minimum temperature for 90 percent coating.

Open-Graded Mix

The method used by the U.S. Forest Service to determine the proportions of open-graded aggregate, emulsified asphalt, and water that will yield a workable paving mixture involves making several trial mixes with varying water and emulsion contents and comparing the characteristics of the mixes. Several trial batches are made for visual inspection. The starting emulsion content may be determined from the CKE test. If the equipment for this procedure is not available, the starting emulsion content can be determined as follows: If the absorption of the aggregate is 1 percent or less, mix the first batch with 5 percent emulsion. If the absorption is 1-2 percent, start with 6 percent emulsion. Start with 7 percent emulsion when the absorption is greater than 2 percent. The starting point for mixing water is determined by adding the minimum amount of water required to darken the oven-dry aggregate. Several batches will be made by holding the emulsion content constant and varying the water content by 1 percent intervals.

The optimum emulsion content and upper and lower limits for moisture content are determined as follows: The minimum acceptable mix must have a moderately thick coating, 90 percent coated area, and fair workability over a range of at least 1 percent moisture. The optimum emulsion content is reached when the coating is heavy, the coated area is 100 percent, and little or no excess fluid is present.

The density of the emulsion-aggregate mixture is determined by preparing a test specimen 102 mm (4 in) in diameter and 64 mm (2.5 in) high by using the kneading compactor.

The report of the laboratory test results includes the same items of information reported for Forest Service dense-graded mixes.

Chevron Method

The Chevron method covers the selection, proportioning, and testing of aggregate, additive, and emulsified asphalt in emulsion mixes. This design procedure is broken into the following parts:

1. Selection of aggregate;
2. Selection of type of emulsified asphalt;
3. Selection of mix proportions;
4. Specimen fabrication;
5. Mix curing;
6. Moisture exposure, including (a) washoff and (b) vacuum saturation;
7. Strength tests, including (a) resilient modulus, (b) resistance R-value, (c) stabilometer S-value, and (d) cohesiometer test;
8. Design criteria; and;
9. Testing schedule.

Figure 2. Testing schedule for EAMs.

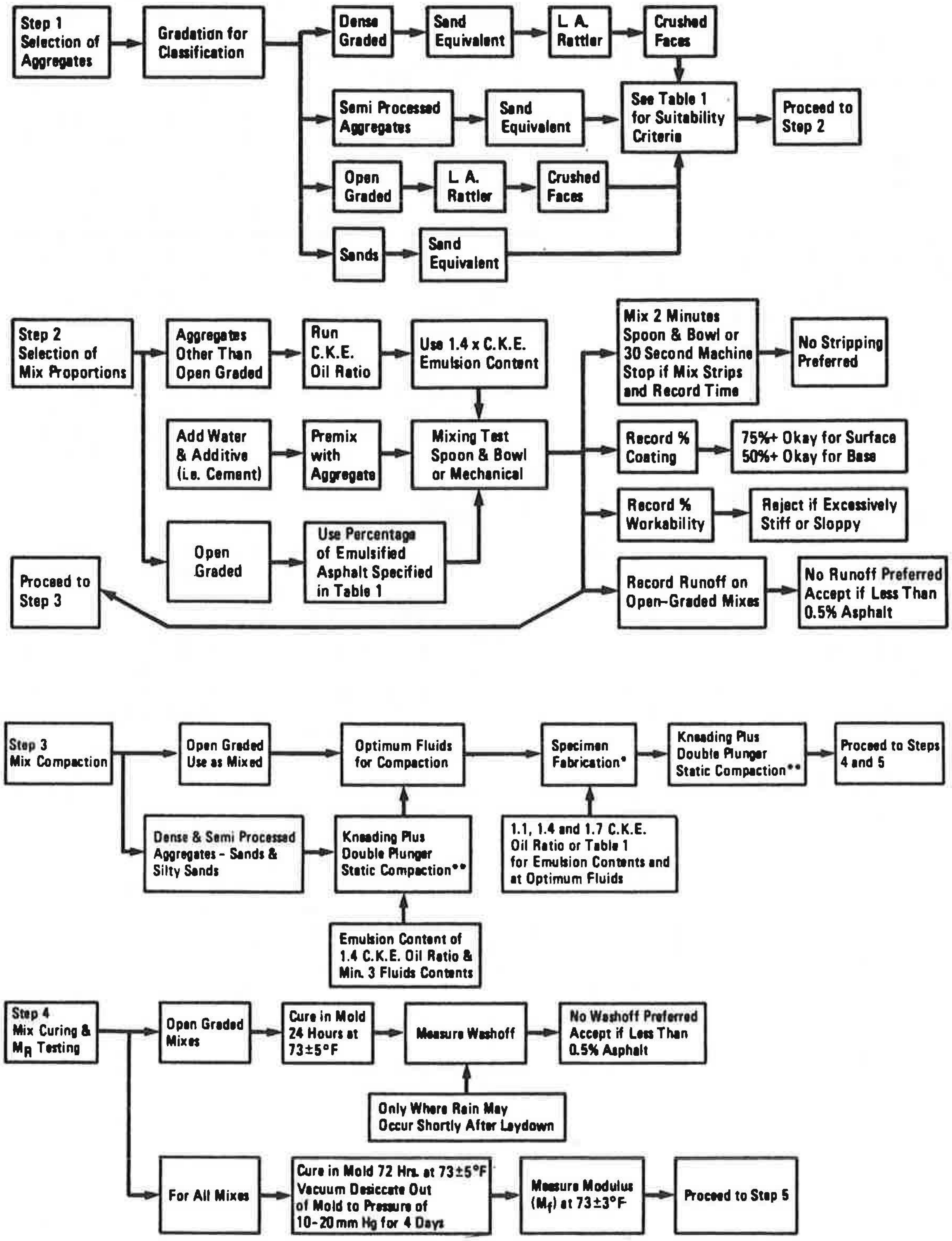
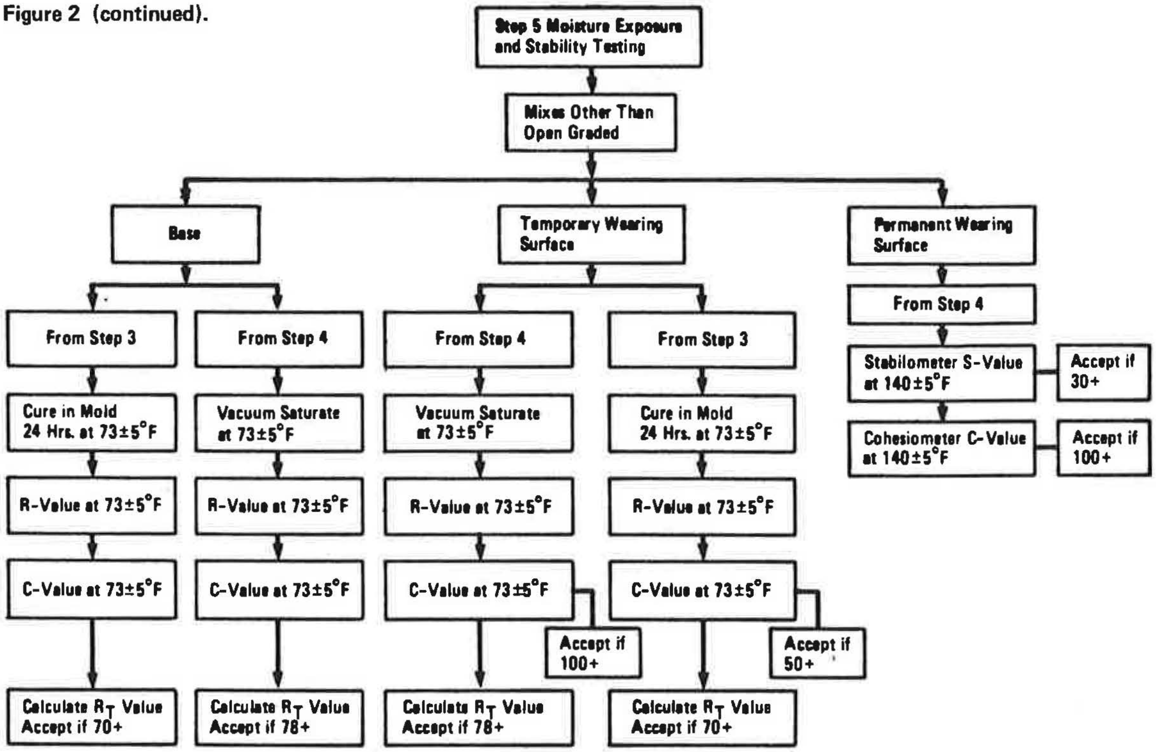


Figure 2 (continued).



*Two specimens prepared at each emulsified asphalt content for all mixes.
**Includes 10-50 blows (250psi) kneading and up to 40,000 lb. double plunger.

Note: t°F = (t°C ÷ 0.55) + 32.

Table 1. Test data recommended for inclusion in Asphalt Institute report on an EAM design.

	Data Item to Be Recorded		
Category	All Mixes	Open-Graded Mixes Only	Not Applicable to Open-Graded Mixes
Aggregate	Gradation Sand equivalent (%) Los Angeles abrasion loss (%) Percentage crushed particles As-received moisture content (%)		CKE oil ratio (%)
Mixture[a]	Compacted mix density[b] (kg/m^3) Asphalt coating (%) Resilient modulus M_R (kPa) Moisture pickup by vacuum soak (%)	Asphalt runoff (%) Asphalt washoff (%)	Resistance R-value Cohesiometer C-value R_t-value Stabilometer S-value Volume of air V_a Volume of asphalt V_b
Mix design	Emulsified asphalt Type Content (%) Residual asphalt content (%) Minimum aggregate premix water content (%) Optimum fluid content for compaction (%)		

[a]Test values by type of mix (dense or open graded) and paving use (base or surface) are reported for each type and content of emulsified asphalt selected for mix design.
[b]Dry vacuum desiccated.

For dense-graded mixes, the starting emulsified-asphalt content is based on the CKE test; for open-graded mixes, the starting emulsion content is selected from a table that is provided with the outline of the test procedure.

Specimens are fabricated at optimum moisture by using a compactive effort similar to that obtainable under field compaction—a light kneading compaction that is followed by a double-plunger static load of 178 kN (40 000 lbf). The mixing fluids for open-graded mixes are assumed to be at optimum for compaction.

The rate at which emulsified-asphalt mixes cure or develop tensile strength is important. A number of factors, including aggregate gradation, type and amount of emulsion, type and amount of additive, and construction and climatic conditions, must be assessed by the engineer in determining the rate of development of tensile strength. To assist the design engineer, strength measurements are made at two curing conditions. The emulsified-asphalt mix is also tested after vacuum saturation.

One of the two specimens fabricated at each asphalt content is cured by placing the mold in a horizontal position

Table 2. Recommended test results and design criteria for Chevron method.

Test Method	Base or Temporary Surface		Wearing Surface	
	Dense-Graded	Open-Graded	Dense-Graded	Open-Graded
Minimum coating (%)	50	50	75	75
Maximum runoff (percentage residual asphalt)	NA	0.5	NA	0.5
Maximum washoff (percentage residual asphalt)	NA	0.5	NA	0.5
Maximum combined runoff and washoff (%)	NA	0.5	NA	0.5
Minimum resistance R-value at 23° ± 3°C				
Initial cure[a]	70	NA	NA	NA
Final cure[b] plus water soak[c]	78	NA	NA	NA
Minimum stabilometer S-value at 60° ± 3°C, final cure[b]	NA	NA	30	NA
Minimum cohesiometer C-value at 23° ± 3°C				
Initial cure[a]	50[d]	NA	NA	NA
Final cure[b] plus water soak[c]	100[d]	NA	NA	NA
At 60° ± 3°C, final cure[b]	NA	NA	100	NA

Note: t°C = (t°F - 32)/1.8.
NA = not applicable.
[a]Cured in the mold for 24 h at 23° ± 3°C.
[b]Cured in the mold for 72 h at 23° ± 3°C plus four days vacuum desiccation at 10-20 mm Hg.
[c]Vacuum saturation at 100 mm Hg.
[d]Applicable to temporary wearing surface only.

for a total of 24 h at a temperature of 23° ± 3°C (73° ± 5°F).

The other specimen is removed from the mold and vacuum saturated for 1 h. This simulates the effect of prolonged exposure to subsurface water on other than open-graded base and permanent wearing surfaces. The ability of the open-graded mix to withstand rain damage is measured by a washoff test.

Resilient modulus (M_R), resistance R-value, and stabilometer S-value are determined on the compacted mixture. M_R is used to measure elastic response and to determine the structural contribution of the mix in the pavement section. The R-value is used to measure the stability or bearing capacity of other than open-graded mixes at a test temperature of 23° ± 3°C (73° ± 5°F). The test is also performed on vacuum-saturated specimens. The S-value is used to measure the stability or bearing capacity of compacted, fully cured, permanent surface mixes other than open-graded. The cohesiometer is used to measure the cohesive resistance or tensile stength of the compacted mixture.

Table 2 provides a summary of recommended test results and design criteria.

FHWA Region 10 Method

Dense-Graded Mix

The FHWA Region 10 procedure for dense-graded mix describes a method of determining the amount of emulsified asphalt to be combined with dense-graded aggregate to produce emulsified-asphalt pavement.

The CKE test is used to determine the "oil ratio", which is multiplied by 1.6 to establish the value for beginning emulsion content. Trial batches of 500 g each are made by holding the emulsion content constant and varying the water content by increments of 1 percent. About 3 percent water is normally used as a starting point. Test specimens are prepared by using a kneading compactor.

Any one of three curing methods can be used, but all involve an air environment at 23°C (73°F). Each method involves a different time period for curing the specimens. The cured specimens are tested for resilient modulus. One of the curing procedures also includes stabilometer and cohesiometer testing. Experimental work is under way to broaden the scope of this test to include M_R measurements after some form of freeze-thaw cycles.

Open-Graded Mix

The FHWA mix-design procedure for open-graded emulsified-asphalt paving mixes was developed for use on projects in Region 10 (Idaho, Oregon, and Washington) and has been used extensively on projects in that area. The procedure describes a method of determining the amount of emulsified asphalt to be used in producing an open-graded mixture by using CMS-2. The beginning emulsion content is determined by the CKE test with appropriate correction for aggregate specific gravity. A determination is then made of the percentage moisture contained in the aggregate. The calculated beginning emulsion content is used with a 500-g aggregate sample. The emulsion and aggregate are mixed for 30-45 s, and the harshness of the mix is observed. A harsh mix will become stiff in the first 15-20 s of mixing, whereas acceptable mixes will not become harsh until they have been mixed for 30-45 s. Any excess liquids that drain from the aggregate must be retained and the weight recorded for later evaluation. The mix should be surface dried, usually with the aid of a fan, after which it is spread in a thin layer and evaluated for the following factors:

1. Thickness of coating, which is evaluated visually and recorded as either thin, moderate, or heavy (T, M, or H);
2. Percentage of particle surface coated, also evaluated visually and recorded; and
3. Any observation that might later be of interest (e.g., an odd smell).

More trial batches are then made by holding the emulsion content constant and increasing the water content by increments of 1 percent. This procedure is continued until a measurable amount of excess liquids can be poured from the mixture into a tared pan. More batches are then mixed at 1 percent above and below the beginning emulsion content. Effective asphalt content is the percentage of residual asphalt in the emulsion minus the percentage of asphalt retained in the tared pans. The film thickness of the asphalt coating (in micrometers) is calculated by using the following formula:

$$\text{Film thickness} = (48.7 \times \text{percentage effective asphalt}) \div \text{surface area (from CKE)} \tag{1}$$

Evaluation of recorded data from the mixing trials will include the following values:

Property	Value
Coating	
Percentage	90-100 percent, absolute lower limit of 85 percent
Thickness	Moderate to heavy
Film thickness	> 20 μm
Excess liquids	Slight amount, 0.1-0.15 percent
Harshness	25 s hand mixing before mix becomes stiff

Armak Method

The Armak Company method of emulsified mix design is a modification of the standard Marshall test procedure, ASTM D1559. Aggregates used in this procedure are dried at room temperature to approximately 1 percent of natural moisture. All aggregate must pass the 12.5-mm (0.5-in) sieve. Mixing and compacting are performed at room temperature. The required amount of additional mixing water is determined by AASHTO method T99. Mixes are usually made from +3 percent to -1 percent of optimum moisture with varying emulsion contents. Normally, five sets of specimens are made and are treated as follows:

1. One set of specimens is tested at room temperature immediately after compaction.
2. A second set of specimens is cured for 24 h at room temperature and then tested at 38°C (100°F).
3. A third set of specimens is cured for 24 h at room temperature and then immersed in a water bath at 38°C for 2 h. These specimens are surface dried with an absorbent material and tested at 38°C for base course and 60°C (140°F) for surface course.
4. A fourth set of specimens is cured for 72 h at room temperature and then tested at 38°C for base course and 60°C for surface course.
5. A fifth set of specimens is cured for 72 h and then immersed in a water bath at 38°C for 2 h. The specimens are surface dried with an absorbent material and tested at 38°C for base course and 60°C for surface course.

McConnaughay Method

K.E. McConnaughay, Inc., of Lafayette, Indiana, has two mix-design procedures for emulsified-asphalt mixes, one for hot mixes and one for cold mixes. With each of these, either the Hveem procedure (ASTM D1560 and D1561) or the Marshall procedure (ASTM D1559) can be used, with the following modifications:

1. Hot mixes--(a) Use ASTM D244, residue by distillation, to determine residue content of the emulsion to be used; (b) weigh the required amount of emulsion on the cold aggregate; (c) mix the emulsion and aggregate and heat on a hot plate with periodic hand mixing at 121°C (250°F); (d) compact the bituminous mixture at 110°C (230°F) in accordance with the design procedure used; and (e) test as prescribed in method used.
2. Cold mixes--(a) Determine the proper emulsion that will provide for satisfactory coating and water resistance by using ASTM D244 coating ability and water resistance and following the alternative provided by note 22 (jobsite aggregate shall be used); (b) determine the residue content of the emulsion to be used by ASTM D244, residue by distillation; (c) determine moisture content of the jobsite aggregate; (d) weigh the required amount of emulsion into the cold, wet aggregate and mix thoroughly; (e) compact the cold emulsion-aggregate mixture in accordance with the design procedure selected, except that compaction is done cold (note that, if the moisture is excessive, it may be necessary to aerate the mixture before compaction); (f) remove the base plate and paper discs and place the mold that contains the compacted specimen on a perforated shelf in a forced-draft oven at 60°C (140°F) for 48 h of curing; (g) after removal from the oven and while the specimen is still at 60°C, apply a static load of 178 kN (40 000 lbf); and (h) test as prescribed in the method used.

Arizona Method

The Arizona Department of Transportation procedure is designed for the testing of specimens made from asphalt emulsions mixed with granular soils. The granular soils to be evaluated are essentially noncohesive, have less than 15 percent passing the 0.074-mm (No. 200) sieve, and have a sand-equivalent value greater than 25. The evaluation is made from test results obtained from specimens formed by use of the Texas Transportation Institute (TTI) compactor and tested by using the Hveem stabilometer and cohesiometer.

The amount of mixing water used should be just sufficient to darken the aggregate. The CKE procedure is used to determine the beginning amount of emulsion. At least three emulsion contents should be used in making test specimens. The emulsion content should be 1.1, 1.3, and 1.5 times the oil ratio obtained in the CKE test. Both the quantity of prewetting water and emulsion content are expressed as a percentage of the dry aggregate weight. Each batch of mixed material should provide a sufficient quantity for three specimens of about 1100 g each plus at least 100 g for determination of total moisture content at the time of compaction.

The TTI compactor is used for molding test specimens. Specimens are molded at ambient temperature. The compaction procedure involves rodding the specimen with a 9.5-mm (0.375-in) diameter bar (mold charged in two layers) followed by compaction with an initial starting foot pressure of 1724 kPa (250 lbf/in²). Initial compaction is continued until the foot penetrates the sample to about 3 mm (0.125 in), which usually requires 10-50 tamps. After initial compaction at 1724 kPa, the foot pressure is changed to 3447 kPa (500 lbf/in²) for 150 tamps. If the material cannot withstand the initial compaction stresses, a double-plunger compaction procedure is used with a load of 178 kN (40 000 lbf) held for 2 min. The specimens are cured in the mold at 25°C (77°F) for three days, after which a set of three specimens can be tested directly by using the stabilometer or, in some cases, undergo a saturation procedure before testing. Vacuum saturation may be used for the soak test. Stabilometer and cohesiometer tests are performed at ambient temperature. Standard calculating procedures are used to obtain (a) compaction density, (b) "cured" and "soaked" test density, (c) cured and soaked moisture content, and (d) cured and soaked values for R, S, and C.

Illinois Method

The Illinois method for design of cold mixtures containing emulsified asphalt and aggregate was developed at the University of Illinois under the sponsorship of the Illinois Department of Transportation and FHWA. Complete details of the method are available elsewhere (8).

The procedure uses a modified Marshall method of mix design and a moisture durability test. The method and recommended test criteria are applicable to base-course mixtures for low-traffic-volume pavements that contain any grade of emulsified asphalt and dense-graded aggregates ≤25 mm (1 in) maximum size. This procedure is recommended for road mixes or plant mixes that are prepared at ambient temperature. The procedure attempts to simulate actual field conditions as nearly as possible.

The design procedure involves the following major steps:

1. Tests are conducted to determine the properties of aggregates and their suitability for use in emulsified-asphalt mixtures.
2. Tests are conducted to determine the properties and quality of emulsions.
3. A simplified procedure is used to estimate a trial residual asphalt content for a given aggregate. This trial

asphalt content is then used in coating tests to determine the suitable type(s) of asphalt emulsion(s) and amount(s) of premixing water required.

4. Mixtures are prepared and aerated to varying moisture contents by using the trial residual asphalt content and the required mixing water. The mixture is then compacted into Marshall specimens, which are dry cured one day and then tested for modified Marshall stability.

5. By using the required mixing water and optimum compaction water content, mixtures are prepared at varying residual asphalt contents. If the optimum compaction water content is lower than the minimum required mixing-water content, aeration is required before compaction. The mixtures are then compacted into Marshall specimens and air cured for three days. The specimens are tested for bulk density, modified Marshall stability, and flow. The moisture susceptibility of the mixture is evaluated by subjecting a series of specimens to a special capillary-water-soak test for four days.

6. The optimum asphalt content is chosen as the percentage of emulsified asphalt at which the paving mixture best satisfies all of the design criteria. The method for calculating the trial residual asphalt content is as follows:

$$R = 0.001\,38\,AB + 6.358\log_{10}C - 4.655 \tag{2}$$

where

R = trial residual asphalt content by weight of dry aggregate (%),
A = percentage of aggregate retained on the 4.75-mm (No. 4) sieve,
B = percentage of aggregate passing the 4.75-mm sieve and retained on the 0.074-mm (No. 200) sieve, and
C = percentage of aggregate passing the 0.074-mm sieve.

Note that gradation is based only on washed-sieve gradations. The R is rounded off to the nearest half percent to yield the trial residual asphalt content.

The initial water content is determined by using the following criteria:

1. For anionic emulsion, the initial trial batch may be mixed without the addition of any water (i.e., in the air-dry condition).
2. For cationic emulsion, a higher water content is often required to produce satisfactory mixes. The coating test should start at about 3 percent water.

Before compaction, the mix is placed no deeper than 25 mm (1 in) in an aeration pan. The pan with the mixture is placed in a curing oven at 93° ± 3°C (200° ± 5°F). The mixture is stirred and weighed every 15 min until the weight is within 20 g of the required weight loss. The mixture is then cooled to 22° ± 1.7°C (72° ± 3°F). The mixture is stirred every 10 min until the calculated required water loss is complete. It is then ready for compaction. The mixture is compacted in the Marshall mold by using 75 blows on each side of the specimen. The specimens are cured at 22° ± 1.7°C in the forming mold for a specified curing period of 24-72 h. The specimens must be set on their edges for equal ventilation on both sides. The specimens are removed from the mold approximatey 2 h before the intended testing time and brought to a temperature of 22° ± 1.7°C. The testing load is applied at a constant rate of deformation of 51 mm/min (2 in/min) until failure. Three companion samples are placed in a capillary soak test: The specimens are placed in a modified mold in water at 22°C (72°F) to a depth of 25 mm for 48 h and are then removed and extruded from the modified molds and tested in the same way as the unsoaked specimens.

The test results are plotted on graphs, and the following properties are reported:

1. Dry stability at one day versus compaction moisture,
2. Dry and soaked stability versus residual asphalt content,
3. Dry bulk density (corrected for moisture) versus residual asphalt content,
4. Percentage total voids versus residual asphalt content,
5. Percentage moisture absorbed versus residual asphalt content, and
6. Percentage stability loss versus residual asphalt content [(dry stability - wet stability) 100/dry stability].

Purdue Method

An investigation conducted by the Joint Highway Research Project at Purdue University, in cooperation with the Indiana State Highway Commission and FHWA, deals with the establishment of a method for preparing and testing asphalt-emulsion-treated mixtures (AETMs) by using Marshall equipment (9). The AETMs were evaluated with emphasis on coating, workability, ease of handling, curing rate, and amount of moisture retained in the mixture before and after compaction. Based on these factors, a method for preparing standard Marshall specimens was developed. In addition, a limited study was conducted to evaluate three different methods for water-sensitivity tests in order to select a satisfactory method for AETMs.

A laboratory investigation to determine the effect of asphalt emulsion content and initial added-moisture content on the design parameters and properties of AETMs by use of Marshall equipment was initiated. The evaluation was conducted at different curing stages of the mix; the early curing condition (one-day air-dry curing) was emphasized. The standard 50-blow Marshall procedure was used.

In addition to the usual Marshall criteria for asphalt mixes, this procedure incorporated two new concepts: (a) Marshall stiffness (S_M), determined as the ratio of Marshall stability to flow, and (b) Marshall Index (I_M), represented by the slope of the linear portion of the load-deformation trace obtained from the autographic Marshall equipment. The autographic equipment provides a continuous recording chart for load versus deformation throughout the testing range.

The initial water content was added to the aggregate, and the mixture was left to stand for 10-15 min before the emulsion was added. Then the materials were mixed with a combination of hand and mechanical mixing. The mixture was then cured in a 60°C (140°F) forced-draft oven for 1 h before remixing and compaction. The AETM was compacted at room temperature by using 50 blows on each side of the specimen. The compacted specimens were left in the mold for about 30 min before extrusion. The samples were then left to cure at room temperature [22°C (72°F)] for the required curing time before testing.

In one phase of the test program, some specimens were oven cured for three days in a forced-draft oven at 49°C (120°F) and then brought to 22°C before testing. The test program also involved soaking specimens for four days in a 22°C water bath before testing in order to measure water sensitivity.

Perhaps the most significant finding of this research is that two different curing periods will provide better understanding and control of mix performance. The two curing periods must be selected to represent the early curing condition and curing for relatively long periods, and emphasis must be placed on the AETM properties in the early curing condition. Furthermore, more reliance on the use of water-sensitivity test results (soaked specimens) as opposed to dry test results are beneficial in providing realistic results and better control of AETM properties. This research also verified some of the effects on AETM properties of asphalt emulsion content, percentage of added water, and curing time.

The test results show that a high degree of stability is attained at the expense of lowered durability (measured as the resistance to water damage). The research indicates that the final design must provide a balance between stability and durability requirements. This can be achieved by controlling and evaluating both the dry and soaked properties of the mix and putting greater emphasis on the soaked specimens.

SUMMARY AND CONCLUSIONS

The available literature on the development of mix-design procedures for emulsion-aggregate mixtures indicates a multiplicity of approaches. There appears to be no consensus concerning the determination of mixing-water requirements, optimum emulsion content, degree and method of curing, specimen formulation, or stability (strength) criteria. However, the following general conclusions can be drawn from test procedures currently in use:

1. Most of the known methods for the design of emulsion-aggregate mixtures use Hveem or Marshall test equipment and include some type of modification(s) to the procedure in relation to specimen preparation, curing, and test temperature.
2. It is usually necessary to add additional water to an emulsion-aggregate mixture to aid in mixing and coating. The amount of water is often determined by trial and error, based on visual inspection of the degree of coating and the amount of runoff.
3. Most procedures use the CKE test to determine the starting percentage of emulsified asphalt. Then mixes that use emulsion percentages above and below the starting percentage can be made for evaluation.
4. The method of curing has a significant effect on the results obtained. In some procedures, a curing or aeration period precedes the molding of the specimen; in others, curing of the molded specimen is required.
5. There are no standard acceptance criteria for EAMs. Acceptance criteria are based on the specific design method used. Different procedures may produce different test values for the same mixture.
6. Complete coating of all aggregate particles is not necessary for an EAM to perform satisfactorily.
7. The evaluation of open-graded EAMs is based primarily on coating, film thickness, workability, and runoff.
8. Considerably more work needs to be done to correlate laboratory test values with field performance characteristics, particularly with respect to the curing of the mixture.

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Use of Marshall Equipment in Development of Asphalt Emulsion Mixture Design Methods and Criteria

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Design procedures for emulsified-asphalt mixtures have been developed by using Marshall equipment. The procedures are intended for use with dense-graded aggregates in base courses on low-volume roads in Illinois. Laboratory and field tests conducted to provide a basis for selecting strength tests and criteria, curing times and temperatures, moisture absorption, and durability tests and criteria are described.

A mix-design method for dense-graded asphalt emulsion cold mixes that uses Marshall equipment has been developed. Details on the design procedure are available elsewhere (1-3). This paper describes why certain tests, curing times, mixing procedures, and stability criteria were selected.

The design procedure was developed specifically for base courses for low-volume roads in Illinois. The mixtures typically use local dense-graded gravel-sand or crushed-limestone aggregates. Several cities and counties in Illinois have used such asphalt emulsion bases on low-volume roads with generally good success. For example, Clark County has constructed more than 322 km (200 miles) of such bases in the past 15 years. Only a small amount of localized repair has been necessary on these pavements, where (a) the base thickness or subgrade stability or both were deficient and (b) construction problems, such as compaction before adequate aeration, were encountered.

This paper outlines mix-design requirements, describes structural and durability criteria, and finally gives some mix-design applications.

MIX-DESIGN REQUIREMENTS

A design procedure for an emulsified-asphalt mixture (EAM) must determine the following:

1. The suitability of aggregates and emulsified asphalt,
2. The compatibility of emulsified asphalt and aggregate,
3. The optimal moisture content for compaction,
4. The optimal residual asphalt content, and
5. The adequacy of structural and durability properties.

Based on results from field and laboratory studies in Illinois (4-6) and other studies (7-16), the following design criteria are considered important in selecting the optimal residual asphalt content:

1. An EAM must provide adequate stability when tested in a "soaked" condition to provide adequate resistance to traffic load during wet seasons. Considerable free moisture is available in Illinois. Most subgrade soils drain poorly, and most low-volume roads are constructed with poor drainage characteristics (i.e., shallow side ditches and a high water table).
2. The percentage loss of stability of the EAM when tested soaked as opposed to dry should not be excessive. A high rate of loss indicates that the EAM has a high susceptibility to moisture and that softening and disintegration may occur during wet seasons.
3. The total voids in the EAM should be within a specified range to prevent either excessive permanent deformation and moisture absorption (for too high a void content) or bleeding and excessive cost of the residual asphalt from the EAM (for a low void content).
4. Moisture absorption into the EAM should not be excessive so that the potential of stripping or weakening of the bond between the residual asphalt and the aggregate is minimized.
5. Residual asphalt should provide adequate coating of the aggregate and should be resistant to stripping.

The basic design philosophy is that the residual asphalt content selected should meet all of these criteria and maximize the soaked stability of the mixture.

STRUCTURAL AND DURABILITY CRITERIA

A dense-graded EAM exhibits a wide range of structural and durability properties. This is generally attributable to the wide range of aggregates used for EAM bases but may also be attributable to the variety of types and grades of asphalt emulsion that are available. EAM material proportions can be optimized for proportioning, mixing, and compaction by using the mix-design procedure. However, even such an optimized mix may still not be structurally adequate or have sufficient durability. Criteria and tests are needed to determine the adequacy of an EAM for use in pavement bases.

A structural analysis of cold asphalt mixtures used in pavement bases was conducted as part of this study (7). The primary types of distress that occur in cold EAM pavements include alligator cracking (or repeated-load fatigue), rutting, and mix disintegration. Many of the problems are associated with a loss of subgrade support caused by the poor drainage conditions typical of low-volume roads. In the relatively wet climate of Illinois, there is an excess of soil moisture throughout most of the year. Thus, there is ample opportunity for free moisture to collect in the EAM base course, and the effect of moisture on the durability of EAMs must be considered.

The objective of the structural analysis was to develop procedures that relate cold EAM structural properties—specifically, the structural coefficient—to pavement performance. The major structural property of the cold-mix material that was correlated with performance was the resilient modulus M_R. Other structural properties, such as fatigue and permanent deformation (rutting) under repeated load, are strongly related to M_R. In base courses, the stiffer the EAM, the less potential there is for fatigue or alligator cracking and rutting distress to occur (8). A stress-dependent finite-element model of pavement analysis was used to analyze critical deflections, stresses, and strains over a range of material types and properties (nonstabilized granular, cold asphalt mixtures, and hot asphalt mixtures). The validity of the approach and the correlations was established by using data and results from the AASHO Road Test and other studies. Curing of the cold-mix base after compaction was found to be a critical factor. The better the curing environment (i.e., high temperature, dry weather, nonsealing of the base), the greater is the potential for improved performance.

These results were used to establish a correlation between M_R and the structural coefficient of the EAM base. By using data from the laboratory study, a correlation was also established between the M_R and Marshall stability at ambient temperature. Thus, the Marshall stability of the EAM could be used to determine the structural coefficient for design. By using the Illinois Department of Transportation (DOT) design procedure, a thickness of EAM base can be determined that provides a pavement that has a specified design life. As long as the structural properties of the EAM are retained, the pavement should provide the required service. If, however, the EAM base softens or distintegrates as a result of durability problems, the pavement will experience premature distress. To avoid this possibility, structural and durability criteria have been established.

The durability of EAM in relation to moisture and temperature effects was studied. Data for Marshall stability and M_R versus curing time were developed for air curing, capillary soaking, vacuum soaking, delayed capillary soaking, and delayed vacuum soaking. The effects of high-temperature soaking and freeze-thaw were also investigated.

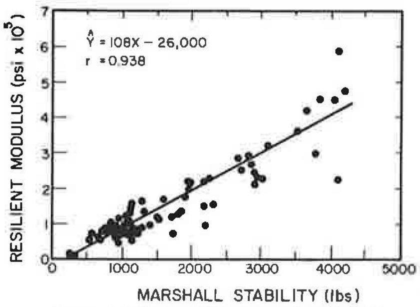
Materials and Testing

The aggregate used consisted of pit-run gravel from Clark County, Illinois, which has been successfully used in many kilometers of EAM. The emulsion used in the experiment was a CMS-2. The mix-design procedure and material properties are described elsewhere (1). Liquid content for mixing was 5.8 percent (4 percent residual asphalt by weight of dry aggregate). The total liquid content was satisfactory for immediate compaction.

Marshall-sized specimens were prepared. All specimens remained in the compaction molds until just before testing. This not only made handling of the specimens easier but also provided confinement (similar to the field condition) to eliminate possible damage from unconfined swell. A total of 74 specimens were prepared in this way. This established the four basic groups of the experiment: air cure, capillary soak, vacuum soak, and delayed capillary and vacuum soaks. Two structural measurements were selected for the study: diametral resilient modulus and Marshall stability (6).

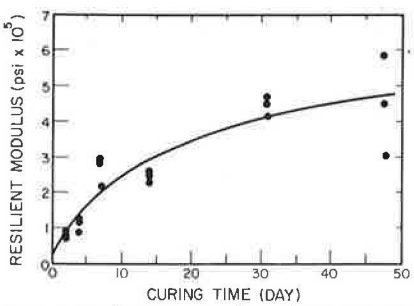
At the designated time, each set of specimens was ejected from the molds, a bulk (wet) density determination was made, and then diametral resilient modulus and Marshall tests for stability and flow were conducted at 22.2°C (72°F). After each set was tested, the samples were pulverized and mixed together to obtain the average moisture content of each set at the time of test. To determine moisture content, the samples were dried in a forced-draft oven at 127°C (260°F) for a minimum of 24 h. The moisture contents were used to calculate dry density, percentage saturation, voidless mix density, and percentage

Figure 1. Correlation between diametral resilient modulus and Marshall stability for EAM.



Note: 1 lbf/in^2 = 6.896 kPa; 1 lbf = 4.45 N.

Figure 2. Effect of air curing on resilient modulus of EAM.



Note: 1 lbf/in^2 = 6.896 kPa; t°F = (t°C ÷ 0.55) + 32.

air voids for each sample. Percentage voids was calculated as specified by Darter and others (1).

Soaking Procedures

The capillary soaking procedure used in the experiment is described elsewhere (6). Vacuum saturation of the samples was accomplished by using the equipment and procedures described in ASTM C593. After vacuum saturation, the samples were kept immersed in the same room-temperature water bath as the capillary samples until time for testing.

Strength Correlations

In a previous study (6), correlations between diametral resilient modulus, indirect tensile test, and Marshall stability test were established. The relation between the resilient modulus and Marshall stability takes on important significance where Marshall equipment is used in the mix design, since it can provide direct input to thickness design by means of relatively simple and inexpensive equipment. In this experiment, all strength determinations (diametral resilient modulus and Marshall stability) were conducted at room temperature [22.2°C (72°F)]. Figure 1 shows the correlation results of resilient modulus versus Marshall stability. The correlation coefficient for all 74 samples in this experiment was 0.938. For practical use, a resilient modulus approximately 100 times the Marshall stability value, measured at 22.2°C, would provide a reasonable estimate.

Air Cure

The air-cure group of specimens was tested at various times: from immediately after compaction to 48 days of air curing at room temperature. The moisture content ranged from an initial 5.8 percent to approximately 0.6 percent, and the release of moisture occurred rapidly during the initial days of curing. Figure 2 shows the plotted results of the air-cured resilient modulus, tested at 22.2°C (72°F), which indicate a rapid initial increase in strength, from approximately 96.5 MPa (14 000 lbf/in^2) to more than 2757.6 MPa (400 000 lbf/in^2). The rapid initial increase in strength is considered to be directly related to the rate of moisture reduction in the EAM. Under extended air curing, beyond that shown, the strength is expected to continually increase because of the "hardening" of the residual asphalt with time. In general, laboratory air curing at constant temperature from the top and bottom surfaces of a 63.5-mm (2.5-in) thick specimen would be expected to be more rapid than curing under field conditions at the same temperature.

Strength gains have been documented under actual field conditions and are illustrated in a figure later in this paper. A laboratory study by Schmidt and Graf (9) of the effects of moisture on asphalt-treated mixes has shown that the resilient modulus is reversible. There is significant loss in strength when the material is saturated and a corresponding increase in strength upon drying. The cyclic wetting and drying imposed on the samples produced corresponding cyclic strength properties, and successive cycles showed a trend toward continual strength increase at given moisture contents. It appears that the dry-wet cycles become less severe. In other words, the drying cycles result in a beneficial increase in strength that is greater than the damage done during the wetting cycles. Although the study by Schmidt and Graf (9) involved vacuum saturation and vacuum desiccation of laboratory samples, similar moisture behavior can be expected in the field during fluctuations in such factors as the groundwater table, rainfall, infiltration, capillary action, and temperature gradient. Moisture contents of EAM cores taken from 27 different projects in seven states (10)—projects that exhibit a wide variety of gradations, densities, and other mix characteristics—range from nil to as high as 10.1 percent. For EAMs that were similar in gradation and density to that used in this study, moisture contents between approximately 1 and 2 percent were common, even four years after construction.

In view of the above, one might ask the following questions: Isn't vacuum saturation too severe, since it does not represent field conditions? Is there a difference in structural response based on the soaking method? Is there a net benefit in the drying cycle over the wetting cycle?

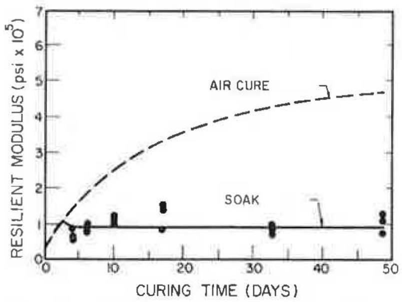
Capillary and Vacuum Soaking

Figures 3 and 4 show the results of resilient modulus versus time, tested at 22.2°C (72°F), for the two soaking methods used in this experiment. In both cases, the compacted specimens were allowed to air cure for three days before soaking was begun. It can be seen that the strength gain ceased in both cases and remained dormant during the soaking period. A statistical analysis that compares the data for the capillary-soaked versus the vacuum-soaked specimens shows no significant difference for the resilient moduli or for stability. It is inferred from this study, therefore, that the effect of capillary soaking may be as severe as that of vacuum soaking.

In an earlier study by Terrel and Monismith (11), a modified wet-sand apparatus was used to investigate the effects of moisture. These results show similar behavior; i.e., the gain in strength stopped or decreased slightly with time. It would appear, therefore, that each of the soaking procedures produces similar results. Even though the vacuum method resulted in higher moisture contents and higher degrees of saturation than the capillary method (see Figure 5), the effect on strength was the same.

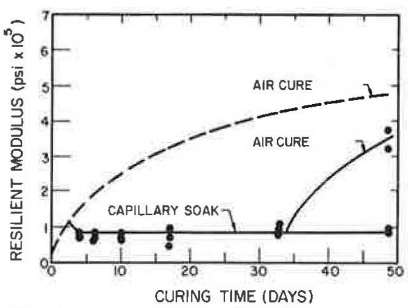
Figure 5 shows the rapid change in moisture content associated with either test method. Figure 5 also explains the rate of strength increase when a set of specimens was removed from capillary soaking and allowed to air cure. The strength increase as moisture is lost is similar to the original air-curing curve. Scrimsher and others (17) conducted a study in which they used four moisture tests:

Figure 3. Effects of vacuum soaking on resilient modulus of EAM.



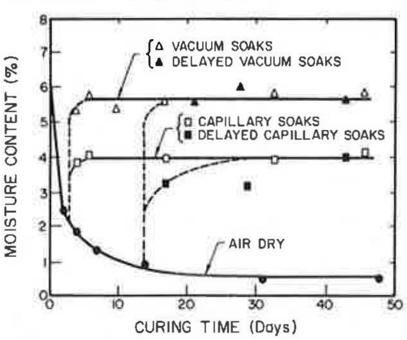
Note: 1 lbf/in^2 = 6.896 kPa; t°F = (t°C ÷ 0.55) + 32.

Figure 4. Effects of capillary soaking on resilient modulus of EAM.



Note: 1 lbf/in^2 = 6.896 kPa; t°F = (t°C ÷ 0.55) + 32.

Figure 5. Effects of EAM moisture contents for various curing methods.



moisture vapor saturation, immersion swell, sand bath assembly, and capillary absorption. That study showed clearly that the capillary absorption test has the following advantages over the other tests:

1. There is realistic representation of field conditions for a base course.
2. Samples are confined and thus do not "fall apart" during soaking.
3. Simple, inexpensive equipment is used.
4. Absorbed moisture content can be easily measured.

Delayed Capillary Soaking and Delayed Vacuum Soaking

In the delayed-soaking test groups, all sets were allowed to air cure for 14 days before the soaking procedure began. Figures 6 and 7 show the results [all tests at 22.2°C (72°F)]. In both cases, there was a rapid loss in strength that either leveled off or continued to decrease gradually. Once again, there was no significant difference in the strength values for the two soaking methods even though the moisture contents and degrees of saturation (see Figures 5 and 8) are different. Statistically comparing delayed-soak samples with soaked samples for similar "soaking times" showed significantly higher strength values in the delayed-soak samples. Thus, it is inferred that the detrimental effects of moisture are lessened as air curing is increased or the extent of breaking of the emulsion is increased. This points up the extreme importance of obtaining adequate early field curing.

One set in the delayed vacuum-soaked group was subjected to three days in a 60°C (140°F) water bath and then four additional days of soaking at 22.2°C (72°F). The results shown in Figure 7 indicate an increase in damage as a result of the elevated soaking temperature.

Another set from the delayed vacuum-soaked group was subjected to three freeze-thaw cycles, each of which consisted of 24 h in a -28.9°C (-20°F) freezer plus 24 h in a 22.2°C water bath. The six-day freeze-thaw sequence was preceded and followed by three days and four days, respectively, of immersion in a 22.2°C water bath. Once again, it should be noted that all samples remained in their molds until just before testing so that swell was restricted to two dimensions. As Figure 7 shows, the strength loss was substantial. Admittedly, this was an extreme test procedure. However, considering the similar damage caused by the different methods of soaking, plus the fact that the difference in total moisture content between vacuum saturation and capillary saturation was less than 2 percent (or approximately 20 percent less in degree of saturation), the possibility of field damage from freeze-thaw seems great. The fact that asphalt cement and emulsions "waterproof" the mixtures should not prevent further study into the effects of freeze-thaw, particularly in a system that incorporates water in its construction.

Figure 9 compares all of the results from the durability study.

SELECTION OF LIMITING CRITERIA FOR MIX DESIGN

Specific durability and structural tests and limiting criteria that relate realistically to field conditions and performance must be selected for mix design.

Structural Test

Significant correlations between the major structural tests of resilient modulus, Marshall stability, and indirect tensile strength (6) indicate that any one of these could be used to provide a structural evaluation of the EAM. Marshall stability is selected because it is the standard test used by the Illinois DOT for routine hot-asphalt mix design and the equipment is readily available.

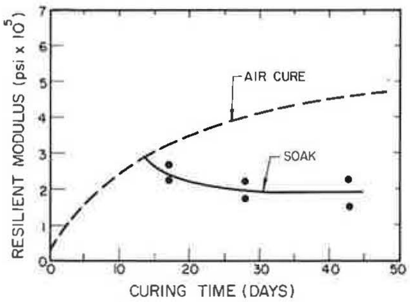
Durability Test

The capillary absorption (or soak) test is believed to be the most realistic test available that represents the field moisture conditions of an EAM base course. Extensive use and evaluation of the test have shown it to be very simple, convenient, and realistic (6, 17). The only disadvantage is the relatively long soaking time required. Based on experimental testing, a four- to five-day soak is believed to be adequate to provide a realistic indication of the moisture durability of the EAM.

Minimum Marshall Stability

An EAM must have at least minimal Marshall stability to prevent excessive permanent deformation of the base course. The results for field cores taken from Clark County, Illinois, are shown in Figure 10. The base EAM ranged in thickness from 102 to 203 mm (4-8 in). The 203-mm cores were cut in half and tested separately as top and bottom. The results show an increase in stability with

Figure 6. Effects of delayed capillary soaking on resilient modulus of EAM.



Note: 1 lbf/in² = 6.896 kPa; t°F = (t°C ÷ 0.55) + 32.

Figure 7. Effects of several curing conditions after delayed vacuum soaking on resilient modulus of EAM.

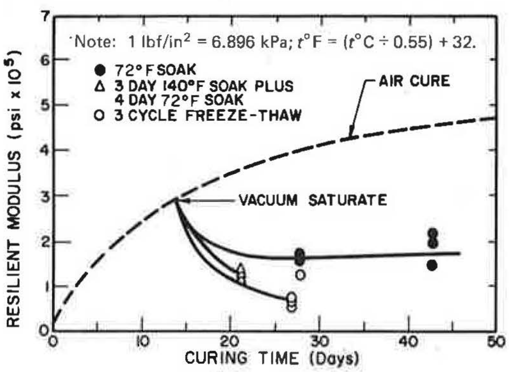


Figure 8. Resilient modulus versus degree of saturation for various curing methods.

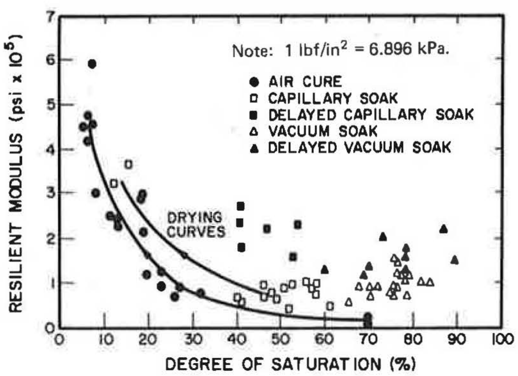
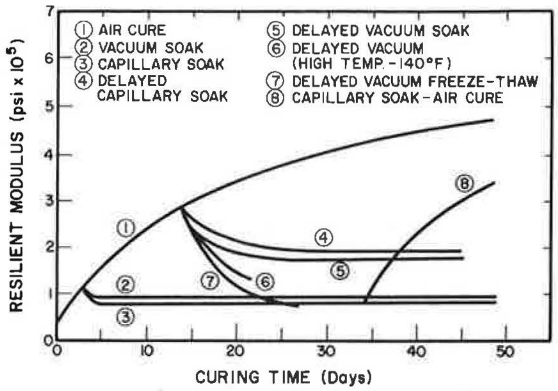
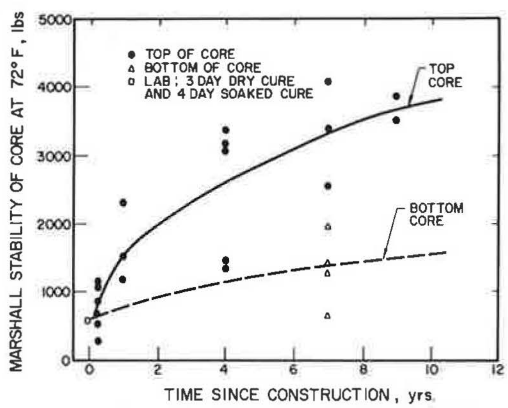


Figure 9. Resilient modulus of EAM as affected by various curing conditions.



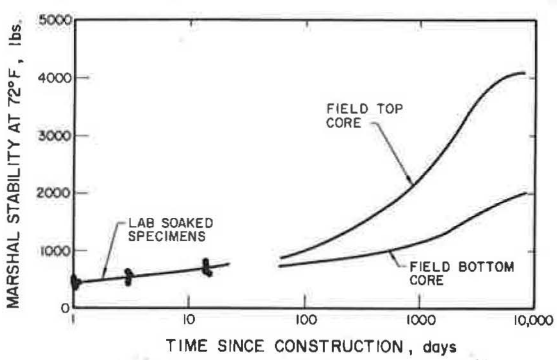
Note: 1 lbf/in² = 6.896 kPa; t°F = (t°C ÷ 0.55) + 32.

Figure 10. Marshall stability of cores cut from EAM projects in Clark County, Illinois.



Note: t°F = (t°C ÷ 0.55) + 32; 1 lbf = 4.45 kN.

Figure 11. Comparison of effects of field and laboratory curing on stability of EAM.



Note: t°F = (t°C ÷ 0.55) + 32; 1 lbf = 4.45 kN.

time after construction. The bottom of the cores shows significantly less stability than the top. This is the result of (a) moisture preventing the complete breaking of the emulsion, (b) moisture causing a stripping of the asphalt and weakening of the asphalt-aggregate bond, or (c) moisture never being fully released (cured) from the system and resulting in stagnation of strength gain. A comparison of the Clark County field curves (Figure 10) with the laboratory-prepared EAM specimens is shown in Figure 11. The field and laboratory curves appear to approximately meet. These EAM-base pavements have generally performed very well for 10 years.

The following rationale was used in selecting a minimum acceptable stability level:

1. A three-day laboratory dry cure at ambient temperature removes most of the moisture in the EAM specimens and represents a reasonably cured condition (Figure 5). Any heating of the EAM to accelerate the cure has been found to produce a very large increase in stability (i.e., two to five times). Since the EAM is not heated in the field, it is unrealistic to heat it for purposes of rapid curing.

2. A four-day soaking period after the three-day dry curing subjects the EAM to a realistic moisture environment. This gives an indication of the potential moisture durability of the EAM. Results given by Darter

and others (1) show that the loss in stability between the dry and soaked conditions is highly dependent on the content and character of the aggregate fines.

3. The minimal Marshall stability of the EAM after the three-day dry cure and four-day capillary soak should be at least 1.78-2.22 kN (400-500 lbf) for low-volume roads. This limiting criterion is based on both experimental results (such as the Clark County data and other data presented later in this paper for pavements that have performed satisfactorily) and stability correlations with the base structural coefficient. According to the correlations presented by Darter and Devos (7), a Marshall soaked stability of 1.78 kN would provide a structural coefficient for design of 0.14, which is equivalent to a crushed-stone base similar to that used at the AASHO Road Test. To provide some margin of design reliability, a minimum of 2.22 kN is desirable. The various EAMs described by Darter and others (1) have a soaked stability greater than 2.22 kN and have shown reasonably good field performance.

Loss of Stability

The extent of loss of stability resulting from the four-day capillary soak test is believed to be an indication of the potential durability of the EAM. A 50 percent loss is considered to be the maximum acceptable. EAM mixtures that had greater than 50 percent loss of stability had high clay contents, low sand equivalents, and low residual asphalt contents. As asphalt content is increased, loss of stability decreases significantly.

Compaction

Laboratory results showed that increasing the compactive effort from 50 to 75 blows increased stability, resilient modulus, and dry bulk density (6). The total voids and moisture absorbed were decreased. Field results are shown in Figure 12, where density of EAM base cores increases with time since construction. The mean data points for 50- and 75-blow laboratory compaction are also shown. The 75-blow density represents field density after about one to two years. It is believed that field compaction should be required to achieve at least 95 percent of 75-blow laboratory compaction.

Percentage Moisture Absorption

The more moisture that enters the EAM, the greater is the potential moisture damage. The California procedure that uses the capillary absorption test limited moisture absorption to 5 percent. After examination of the results from many mix designs, it is believed that a slightly lower value of 4 percent should be set as a limiting value.

Percentage Total Voids

The total voids (air plus water) affect the density, the amount of water that may be absorbed into the mix, and the potential for permanent deformation or rutting of the mix. A range of 2-8 percent for base course is recommended by the Asphalt Institute (18), but no information is available to support this range. It may not be necessary to place restrictions on voids, since absorbed water correlates well with void content (6).

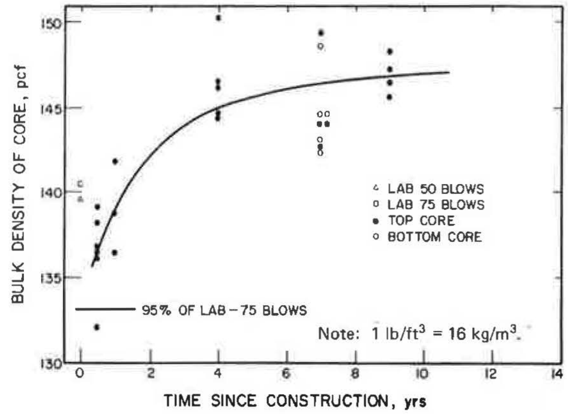
MIX-DESIGN APPLICATIONS

The aggregates used were obtained from pits and quarries in Illinois and are representative of typically used pit-run gravels and crushed-limestone aggregates.

Bond County Gravel

Bond County gravel comes from a pit located near the project and, if acceptable, would provide considerable economic advantage over other aggregate, which would have to be hauled in. The aggregate and emulsion properties are given by Darter and others (1). The washed gradation reveals a relatively high fines content, which was apparently caused by the failure to strip overburden at the pit. A relatively high amount of clay is indicated by the low sand-equivalent value of 22. Water absorption is excessive, and asphalt absorption may become a problem during later pavement life as asphalt is absorbed into the aggregate and film thickness is reduced. This aggregate would normally be rejected because of the gradation and low sand-equivalent value.

Figure 12. Increase of EAM base density over time from Clark County, Illinois.



The optimal residual asphalt content is estimated to be 5.4 percent, as computed by a prediction equation (1). Coating tests were conducted by preparing several laboratory bowl EAM mixtures over a range of premixing moisture contents (i.e., moisture contained in aggregate before adding emulsion) at the estimated optimal asphalt content. The best aggregate coating was obtained at premix moisture contents of 3-5 percent (excluding water contained in the emulsion). Marshall-sized specimens were prepared with 5.0 percent residual asphalt (actually, 5.4 percent should have been used, since that is the estimated optimal asphalt content) and compacted over a range of moisture contents. The specimens were air cured one day on a laboratory shelf, extruded from their molds, and tested in the Marshall stabilometer at 22.2°C (72°F). A curve that indicates maximum stability occurring at 3.5 percent total moisture content (by weight of dry aggregate) was obtained (see Figure 13). This optimal moisture content at compaction was used in the compaction of all other specimens.

Compacted EAM specimens were then prepared over a range of residual asphalt contents. Specimens were tested after three days of laboratory air curing—called dry curing—for Marshall stability at 22.2°C. Other specimens, after the three-day dry cure, were subjected to five days of the capillary soak test (2.5 days on each side of specimens)—called soaked curing—and then tested for Marshall stability. Dry bulk density and the moisture contained in the specimens were also determined. All results are plotted in Figure 13. Maximum soaked stability occurs at approximately 5.3 percent residual asphalt. There is a large loss of stability between the dry-cured specimens and the soaked specimens, but the amount of the difference decreases with increasing asphalt content. A large loss such as this has only occurred with aggregates that have a low sand equivalent (<25) and a large amount of fines [minus 0.074-mm (minus No. 200) sieve]—i.e., >15 percent.

The residual asphalt content at peak soaked stability is 5.3. The following values of other parameters are obtained from the graphs for this asphalt content:

Mix Parameter	Value at 5.3 Percent Asphalt	Limiting Criterion
Stability loss (%)	57	50 max
Total voids (%)	6.7	2-8
Moisture absorption (%)	3.6	4 max
Modified Marshall stability (kN)	2.4	2.2 min
Aggregate coating (%) (3-5 percent premix moisture)	60-70	50 min

All of the criteria except percentage loss stability are achieved at a residual asphalt content of 5.3 percent. A residual asphalt content of 5.6 percent is required to meet the 50 percent loss requirement. At 5.6 percent asphalt content, all other requirements are achieved. However, the values for soaked stability and absorbed moisture are very close to the acceptable values.

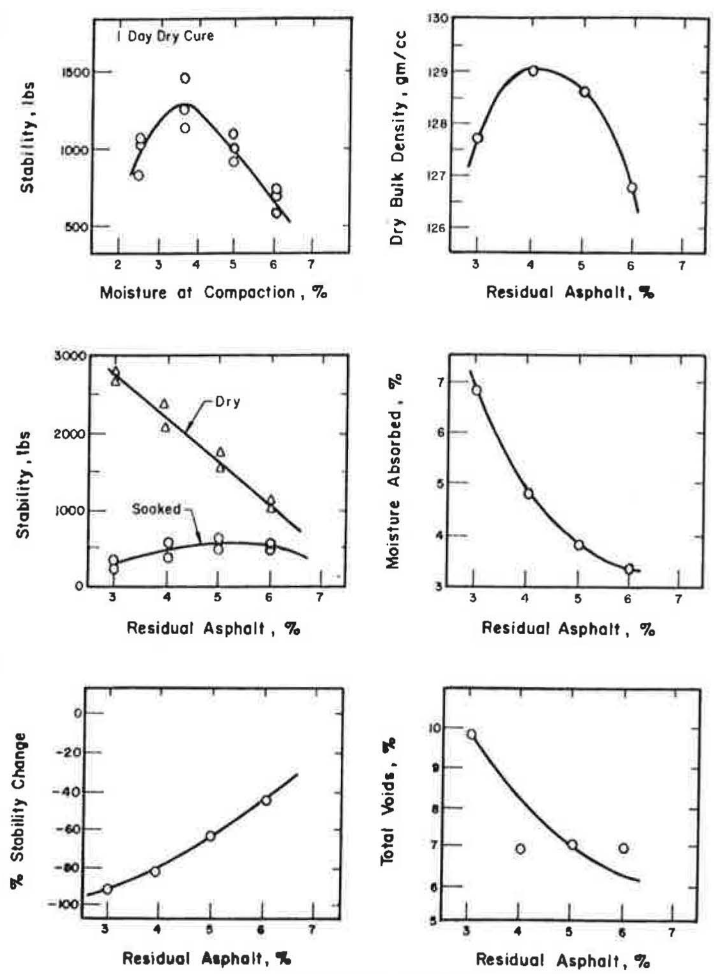
The following recommendations are made for mix design and construction:

1. Residual asphalt content = 5.6 percent by weight of dry aggregate,
2. Asphalt emulsion content (for an asphalt residue of 70 percent) = (5.6/0.70) = 8.0 percent by weight of dry aggregate, or approximately 81.4 L of emulsion per megagram of dry aggregate (19.4 gal/ton),
3. Premixing water content = 3-5 percent by weight of dry aggregate, and
4. Optimal water content at compaction = 3.5 percent by weight of dry aggregate (I.e., total water content in EAM).

The project was constructed in 1976 by using road-mix procedures. Three lift thicknesses were used: 64, 64, and 51.3 mm (2.5, 2.5, and 2.0 in), for a total of 178 mm (7 in). A residual asphalt content of about 4 percent was obtained, which is considerably less than that recommended. The water content of the gravel before mixing was 5.9 percent, which is greater than the optimal range for coating. The field mix observed just before compaction was estimated to have about 60 percent coating. Water content at the first pass of the roller was about 3.9 percent, which is near the recommended 3.5 percent. Field mix was obtained just before compaction and brought to the laboratory in sealed containers for compaction into Marshall-sized specimens and testing. Some of the results obtained are given below (1 kN = 224.8 lbf):

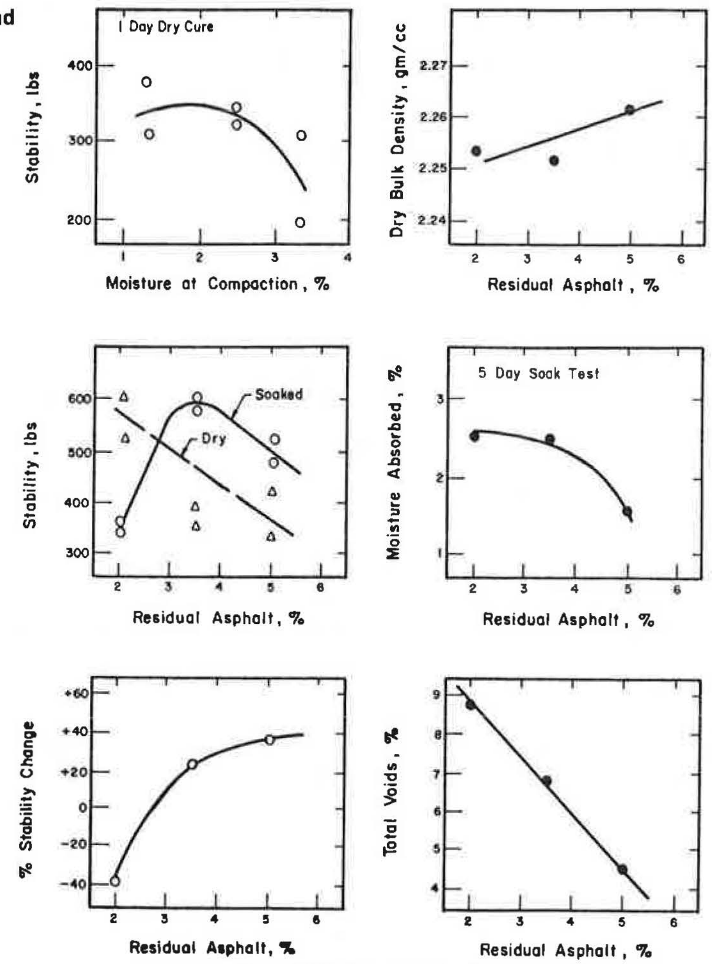
Item	Three-Day Dry Cure	Three-Day Dry Cure plus Five-Day Soaked
Laboratory mix design stability at 4 percent asphalt (kN)	9.78	2.00
Field mix stability at 4 percent asphalt (kN)	6.17	1.3
Moisture content at testing (%)		
Laboratory	2.2	2.6
Field	7.0	7.7

Figure 13. Mix design for Bond County pit-run gravel and HFE-300 emulsion.



Note: 1 lbf = 4.45 kN; 1 g/cm^3 = 0.062 lb/ft^3.

Figure 14. Mix design for Clark County pit-run gravel and HFE-300 emulsion.



Note: 1 lbf = 4.45 kN; 1 g/cm^3 = 0.062 lb/ft^3.

The percentage water absorption for laboratory and field mixes was 4.8 and 5.1 percent, respectively. These data indicate that the field mix has less stability and greater moisture content than the laboratory mixes. This, coupled with the low residual asphalt content, may cause serious problems for the EAM base. After one year of service, the pavement does not show any significant distress.

Clark County Gravel

Clark County gravel has been used in EAM low-volume pavement bases for many years with good success. The aggregate and emulsion properties are given elsewhere (1). The estimated trial residual asphalt content was computed to be 3.7 percent. Coating tests were conducted at 3.7 percent residual asphalt and varying premix moisture contents. Acceptable coatings were obtained over a range of 3-5 percent premix moisture. The total moisture content at compaction (as percentage of dry aggregate) that produces maximum stability is approximately 2.0 percent, as shown in Figure 14. A series of compacted Marshall-sized specimens were prepared and tested dry (after three-day curing on a laboratory shelf), and also after the three-day dry cure and a five-day soak (by using the capillary moisture soak test). Results show a peaked soaked curve with maximum stability occurring at about 3.5 percent residual emulsion. The dry-cure stability curve is greater than the soaked curve at 2-3 percent asphalt content but is less than the soaked curve for higher asphalt contents, as the curve for percentage change in stability reflects. Moisture content absorbed during the five-day soak test and total voids decrease with increased residual asphalt content.

The residual asphalt content at peak soaked stability is 3.5 percent. The following values of other parameters are obtained from the graphs for this asphalt content (1 kN = 224.8 lbf):

Mix Parameter	Value at 3.5 Percent Asphalt	Limiting Criterion
Stability change (%)	+25	-50 max
Total voids (%)	6.8	2-8
Moisture absorption (%)	2.5	4 max
Modified Marshall stability (kN)	2.6	2.2 min
Aggregate coating (%) (3-5 percent premix water)	60-75	50 min

Therefore, all of the criteria are achieved at a residual asphalt content of 3.5 percent. The mixture design used by Clark County uses a slightly coarser aggregate gradation and about 3.5-4.0 percent residual asphalt and has provided successful mixes for more than 10 years.

The following recommendations are made for mix design and construction:

1. Residual asphalt content = 3.5 percent by weight of dry aggregate,

2. Asphalt emulsion content (for an asphalt residue of 70 percent) = (3.5/0.70) = 5.0 percent by weight of dry aggregate, or approximately 50.4 L per megagram of dry aggregate (12 gal/ton),

3. Mixing water content = 3.5 percent by weight of dry aggregate, and
4. Optimal water content at compaction = 2.0 percent by weight of dry aggregate.

CONCLUSIONS

This paper describes the selection of various criteria for the EAM design procedure developed at the University of Illinois. A given mixture should meet the following selected criteria: (a) adequate stability when tested in a soaked condition, (b) no excessive loss of stability when tested soaked as opposed to dry, (c) limited moisture absorption into the mixture, and (d) adequate coating. The basic design philosophy is that a residual asphalt content should be selected that meets these criteria and maximizes soaked stability. Field and laboratory tests were conducted to establish a test series and procedures and tentative limiting criteria for mix design for low-volume bases. Much additional field verification is needed before the procedures and criteria can be used with confidence.

ACKNOWLEDGMENT

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The contents of this paper reflect our views, and we 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 Illinois Department of Transportation or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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Laboratory Evaluation of Asphalt Emulsion Mixtures by Use of the Marshall and Indirect Tensile Tests

MICHAEL S. MAMLOUK, LEONARD E. WOOD, AND AHMED A. GADALLAH

A laboratory procedure for specimen preparation, developed to characterize the asphalt emulsion mixtures used in base courses, is described. The main factors considered in the technique are aggregate coating, workability of the mix, and rate of moisture loss from the mix before and after compaction. The Marshall test was performed at room temperature to evaluate the performance of the mixture. The mixture was further characterized by conducting the indirect tensile test at various temperatures. Both types of tests were conducted for different mix compositions and curing conditions. The specimens were vacuum saturated after different curing times to evaluate the resistance of the mixture to adverse moisture conditions. An evaluation system for asphalt emulsion mixtures is recommended based on the results of the investigation.

Asphalt emulsion (AE) mixes offer several advantages over hot-mixed asphaltic concrete. Asphalt emulsion can be mixed with damp aggregate at ambient temperatures. This is a major advantage in relation to energy saving and air pollution. Either plant mix or road mix can be produced by using this type of mixture. The main disadvantage of AE mixes, however, is the slow development of the strength, which is controlled by the loss of moisture from the mixture.

The performance of AE-treated bases has generally been successful. In recent years, however, some distress has been noted on some heavily traveled roads on which cold-mixed AE-treated bases were used (1). The improper use of AE mixes could be the reason for this distress. The current lack of sufficient data on the influence of different mix components and weather conditions on the behavior of AE mixtures has been instrumental in focusing on the need for a proper design procedure for such mixtures.

Several investigations have been conducted to establish design procedures for cold-mixed AE mixtures, but no standard method has yet been adopted (2). The purpose of this study was to establish a laboratory technique to be used in characterizing cold-mixed AE mixtures used in black bases. Both the Marshall test and the indirect tensile test were used to evaluate the mixture. The susceptibility of the mixture to adverse water conditions was also investigated.

MATERIALS

Aggregate

Two types of aggregate that meet Indiana State Highway Commission (ISHC) standard specifications were used in this study. The first type was a mixture of sand and gravel that consisted of approximately 50 percent calcareous and 50 percent siliceous pieces; 56 percent of gravel particles retained on the 4.75-mm (No. 4) sieve had crushed faces. The second type of aggregate used was crushed limestone. The one aggregate gradation used followed the midspecification of the ISHC #73B gradation band and had a maximum size of 19 mm (0.75 in), as shown in Figure 1.

The properties of these two types of aggregate are given below:

Property	Sand and Gravel	Limestone
Apparent specific gravity	2.71	2.74
Bulk specific gravity	2.61	2.70
Absorption (%)	1.20	1.28

The sand-and-gravel mixture was used in the development of the mix-design procedure and the Marshall test. Both the sand-and-gravel and limestone mixtures were used in the indirect tensile test.

Asphalt Emulsion

The high-float AE used was HFMS-2s (ASTM D977). The physical properties of the emulsion were as follows: (25°C = 45°F):

Property	Value
Saybolt Furol viscosity (s)	>50
Residue by distillation (%)	70
Penetration of residue after distillation, at 25°C, 5 s, 100 g	>200
Specific gravity of residue after distillation, at 25°C	0.986

The compatibility between aggregate and AE was examined according to the ability of the emulsion to coat the aggregate particles. The amount of AE in the mixture was chosen to fall within the ISHC recommended range. Values of 2.5, 3.25, and 4 percent of AE residue by dry weight of aggregate were used.

Figure 1. Aggregate gradation.

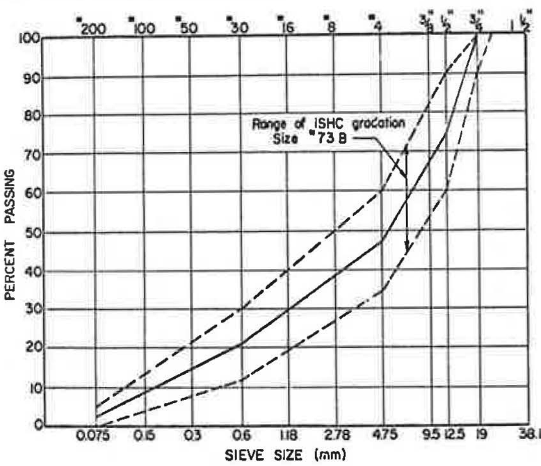
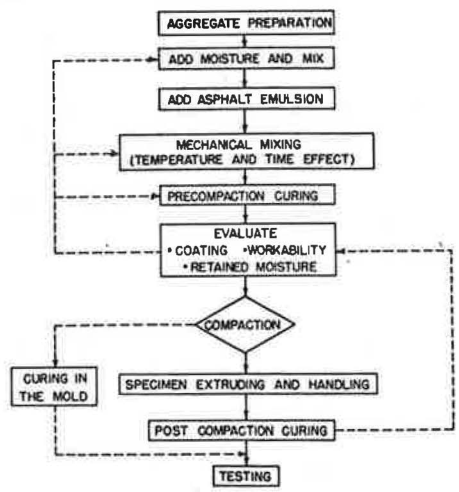


Figure 2. AE mixture preparation and testing procedure.



PROCEDURES FOR SPECIMEN PREPARATION

More effort needs to be expended in controlling and handling AE mixtures than in controlling the traditional hot mixes because more factors in the AE mixture system affect its performance during mixing and specimen preparation. The main factors evaluated to provide an adequate method for preparing and testing the AE mixture specimens were coating of the aggregate, workability of the mix, and the trend of the moisture retained in specimens before and after compaction (the curing rate). The different steps considered in this investigation are discussed below (see Figure 2).

The dry aggregate was blended into 1200-g (2.4-lb) batches by combining the different aggregate sizes to meet the desired gradation. The aggregate was used in the AE mixture at a room temperature of 22°C (72°F). The initial moisture content was added to the aggregate and mixed thoroughly by hand. The purpose of adding moisture to aggregate before mixing with AE is to prevent balling up of fine-grained particles and to provide a uniform AE coating of the aggregate particles. The amount of mixing water that will provide the best coating of aggregate with a certain amount of AE should be selected. A range of 0-4.5

Figure 3. Effect of precompaction curing condition on moisture retained in loose mixture (4 percent added moisture and 4 percent asphalt residue).

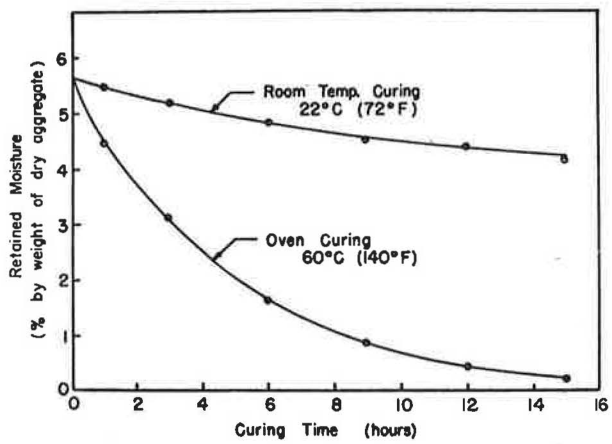
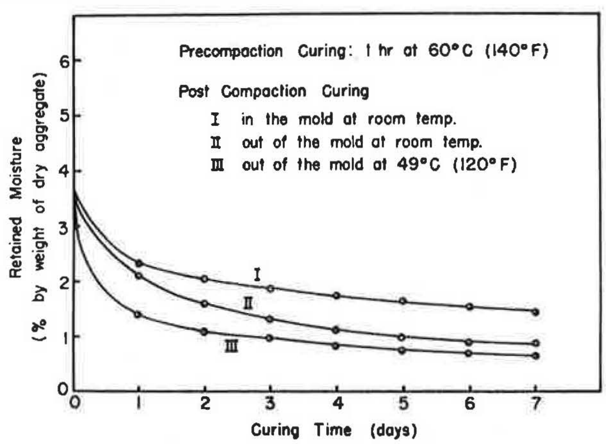


Figure 4. Effect of postcompaction curing condition on moisture retained in compacted specimen (4 percent moisture and 4 percent asphalt residue).



percent of initial added moisture by weight of dry aggregate was used.

The AE was added cold to the wet aggregate and mixed by using a mechanical mixer. Hand mixing was also used to overcome the segregation of fine and coarse aggregates during the mechanical mixing.

Precompaction Curing

The effect of curing on the amount of moisture retained in the mixture before compaction (mix in the loose condition) was investigated. Two conditions of precompaction curing were evaluated: (a) a complete curing of the loose mixture at a room temperature of 22°C (72°F) and (b) curing at 60°C (140°F). The effect of the precompaction curing condition on the moisture retained in the loose mixture is shown in Figure 3.

According to criteria used in Indiana for AE mix preparation, the amount of moisture in the mix should not exceed 4.5 percent by weight of dry aggregate prior to compaction. It was found that this moisture content was obtained after approximately 1 h of curing at 60°C (140°F) when 4 percent initial added moisture was used. The same moisture content was reached after 10 h of curing at room temperature. Since the initial added-moisture content does not exceed this value in most cases, it is recommended that the mixture should be cured for 1 h at 60°C and then remixed for 30 s before compaction.

The precompaction curing process is necessary not only to remove some of the excess water but also to compensate for the high energy provided in the field during mixture preparation. It was also found that precompaction curing provided better coating of the aggregate and easier handling of the mixture than a complete cold process. Precompaction curing produced a mix temperature of about 43°-49°C (110°-120°F) after 1 h at 60°C, which is considered reasonable for cold and intermediate AE mixtures.

Compaction

Marshall specimens 102 mm (4 in) in diameter and about 64 mm (2.5 in) in height were prepared by using 50 blows of the standard Marshall compaction hammer on each side of the specimen. This compacting effort was selected to duplicate the conditions of the pavement in the field under medium traffic. Specimens used in the indirect tensile test, however, were compacted by using a fixed-roller gyratory compaction machine. Twenty revolutions of the gyratory machine at 1.38 MPa (200 lbf/in^2) and a 1° gyration angle were used. Both methods of compaction were found to give similar specimen unit weights.

Postcompaction Curing

Three conditions of postcompaction curing of the compacted specimens were evaluated. The first two conditions were curing in the mold and out of the mold at room temperature. The amount of moisture retained in the compacted specimens was determined for these two cases (see Figure 4). It was found that curing the specimens out of the mold was beneficial in relation to the rate of moisture loss. Out-of-the-mold curing provides more surface area for the moisture to leave the specimen than does curing in the mold.

To expedite the curing process, a third condition was considered: curing the specimen out of the mold at 49°C (120°F). In this case, the amount of retained moisture dropped markedly at the beginning of the curing time and leveled off thereafter (Figure 4). After three days at 49°C, the amount of retained moisture did not exceed 1 percent by weight of dry aggregate. It was concluded, therefore, that curing the specimens out of the mold for three days at 49°C would approximate the long-term curing process in the field.

The levels of postcompaction curing that were considered in the mixture characterization are as follows:

1. One-day air curing at a room temperature of 22°C (72°F),
2. Three-day air curing at a room temperature of 22°C, and
3. Three-day oven curing at 49°C.

These postcompaction curings represented the initial (after construction), intermediate, and long-term curing conditions in the field, respectively.

The problems involved in extruding the specimens from the molds were evaluated at different curing times after compaction in conjunction with the precompaction curing conditions. For the selected precompaction curing condition [1 h at 60°C (140°F) and then remix before compaction], it was found that the specimens could be extruded from the molds without any damage about 30 min after compaction. However, care must be taken in handling specimens of some mix combinations, such as those that have a low amount of AE and/or high water content.

Recommended Method of Specimen Preparation for Marshall Test

Based on the evaluation study, the following procedure is recommended for preparing AE mixture specimens for Marshall testing (3):

1. The aggregate is prepared in approximately 1200-g (2.4-lb) batches, based on the aggregate gradation required.
2. The required amount of initial moisture is added to the cold aggregate and mixed thoroughly by hand.
3. The aggregate-water mixture is allowed to stand for 10-15 min before the AE is added to allow the mixing water to fill the surface voids of the aggregate and to obtain a uniform coating of moisture over the aggregate.
4. The amount of AE that is required to provide a certain AE residue content in the mix is added cold to the wet aggregate and mixed with a mechanical mixer for about 2 min and by hand with a spoon for 30 s.
5. The mix is cured for 1 h in a forced-draft oven at 60°C (140°F) and then remixed for 30 s with the mechanical mixer.
6. The mix is compacted with the mechanical Marshall compaction hammer by using 50 blows on each side of the specimen.
7. The compacted specimens are left in the mold for about 30 min before they are extruded.
8. The specimens are then left to cure for the required curing time and temperature before testing.

CHARACTERIZATION OF AE MIXTURE

Marshall Test

The Marshall test was performed at a room temperature of 22°C (72°F). A continuous chart recording of load versus deformation was obtained from the test (see Figure 5). Modified Marshall stability and Marshall flow were determined. Two new parameters—Marshall stiffness and Marshall index—were also obtained. Marshall stiffness is defined as the ratio between Marshall stability and flow, whereas the Marshall index is the slope of the linear portion of the load-versus-deformation trace. Specimens were tested in both before and after vacuum saturation.

A modified vacuum-saturation method developed by the Asphalt Institute was used in this study (4). According to this method, specimens were subjected to a vacuum of 30 mm Hg for 1 h and then submerged in water for 24 h at room temperature before being tested. A comparative analysis between the dry and vacuum-saturated specimens was performed.

The initial added moisture and its interaction with the AE content proved to have a considerable effect on the modified Marshall stability of the mixture. In addition, the effect of AE content on the stability of the mix was not apparent at early stages of curing, mainly because of the nature of the AE present in the mix at that time. However, the significant effect of AE content became increasingly important during the curing process, at which time the AE residue started to gradually affect the mix properties.

Figure 6 shows the Marshall stability values as a function of AE content and percentage of added moisture. The highest stability values were obtained for samples that had no added moisture (it should be noted that about 0.2 percent moisture content was present in the aggregate). At 1.5 percent added moisture, the highest stability values were obtained for samples that had 3.25 percent AE residue, but the difference in stability was small. When added- moisture content was increased, the samples with the low AE content (2.5 percent) displayed higher stability values.

Total liquid content, which is the sum of the AE residue content and the retained-moisture content, is an important factor in the response of an AE mixture. There exists an optimal liquid content that provides a mix with a maximum Marshall stability value. At a high AE content, a small percentage of initial added moisture is adequate. However, for low AE contents, increasing the amount of added moisture up to a certain limit improves the properties of the mix, as Figure 7 shows. The optimal liquid content at time of testing, after the samples with no added moisture were excluded, was in the range of 4.0-4.5 percent by weight of dry aggregate.

The Marshall stability values for dry and soaked conditions at the three curing periods are shown in Figure 8. A significant result of this test shows that at any curing level the percentage of retained stability increases with increasing AE content in the mix. In addition, the relations between stability and AE content for the soaked samples follow a curvilinear pattern, and an optimal AE content

Figure 5. Relation among Marshall stability, flow, stiffness, and index.

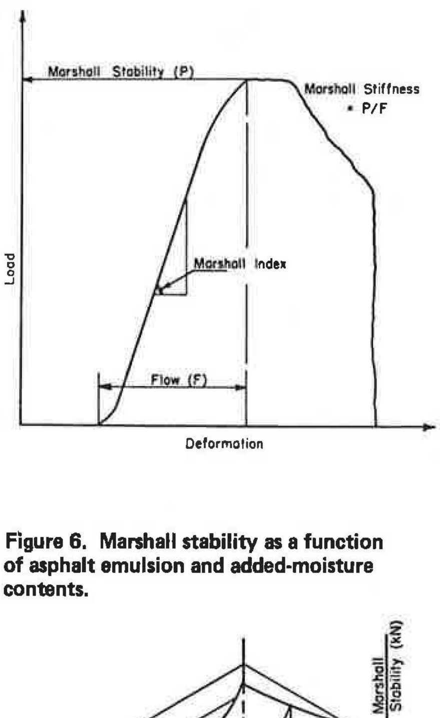


Figure 6. Marshall stability as a function of asphalt emulsion and added-moisture contents.

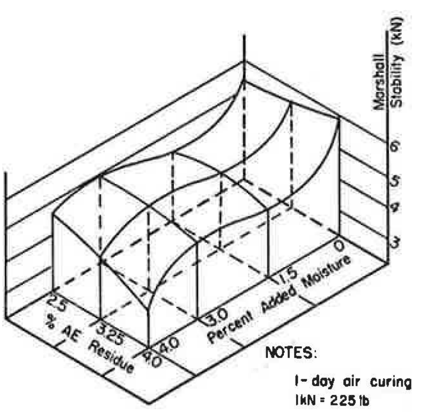
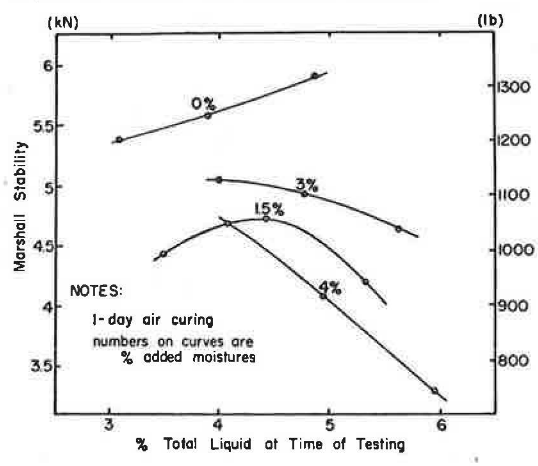


Figure 7. Marshall stability versus percentage total liquid for different added-moisture contents.



value corresponds to a maximum stability value. In contrast, the dry test results showed a decreasing trend with increasing AE content. Longer curing periods for the dry specimens resulted in greater changes in stability as AE content was increased.

The Marshall flow values ranged from 6 to 11 [in 0.25-mm (0.01-in) units]. The flow values increased with increasing AE or added-moisture content. However, mixes with no added moisture exhibited higher flow values than those with 1.5 percent added moisture.

Marshall stiffness and index were both markedly affected by AE content and added-moisture content. Decreasing the AE content resulted in a mix that was less plastic. The slope of the load-deformation curve became steeper, and both Marshall stiffness and index increased (see Figure 9). The same trend held for the effect of percentage added moisture.

Indirect Tensile Test

The indirect tensile test was performed by using the Material Testing System electrohydraulic machine at temperatures of 10°, 24°, and 38°C (50°, 75°, and 100°F). The load was applied at a rate of loading of 51 mm/min (2 in/min) by using two curved, stainless-steel, 12.7-mm (0.5-in) wide loading strips. Continuous recordings of load versus horizontal deformation and vertical deformation versus horizontal deformation during the load application were obtained. Tensile strength, Poisson's ratio, tensile stiffness, and tensile strain at failure were also determined (5,6). The equations used in calculating the above expressions are given elsewhere (7).

In this part of the study, both sand-and-gravel and limestone mixtures were used. Two initial added-moisture contents (3 and 4.5 percent) and two AE residue contents (3.25 and 4 percent) were evaluated. Both one-day air curing and three-day oven curing were investigated.

The tensile strengths, which ranged between 25 and 596 kPa (3.6 and 86.4 lbf/in^2), were largely affected by test temperature, aggregate type, curing, and initial added-moisture content. High tensile-strength values were obtained at low test temperatures with limestone mixtures, three-day oven curing, and small initial amounts of added moisture. The interaction effect of aggregate type, curing, and test temperature on tensile strength is shown in Figure 10.

Poisson's ratio was very sensitive to test temperature. When the test temperature increased, Poisson's ratio increased. At a test temperature of 38°C (100°F), the specimens began to develop hairline cracks before total failure. Thus, values of Poisson's ratio greater than 0.5 were obtained. In addition to test temperature, high values of Poisson's ratio were obtained at one-day air curing in comparison with three-day oven curing. In the rest of the analysis, Poisson's ratio was assumed to be 0.3, 0.35, and 0.4 at temperatures of 10°, 24°, and 38°C (50°, 75°, and 100°F), respectively.

The effect of test temperature, aggregate type, and AE content on tensile stiffness is shown in Figure 11. The test

Figure 8. Marshall stability for dry and soaked specimens after different curing periods.

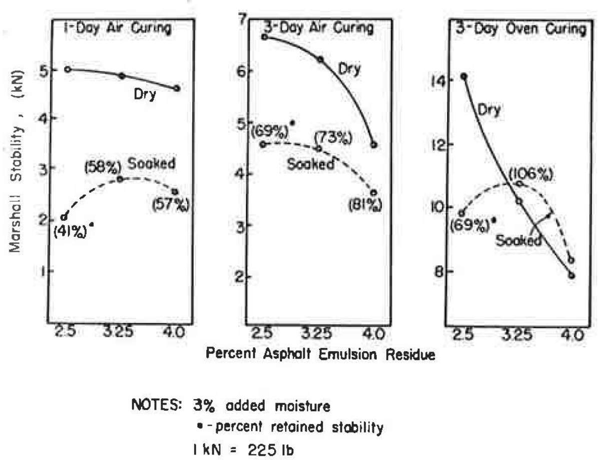


Figure 9. Asphalt emulsion content versus Marshall stiffness and index.

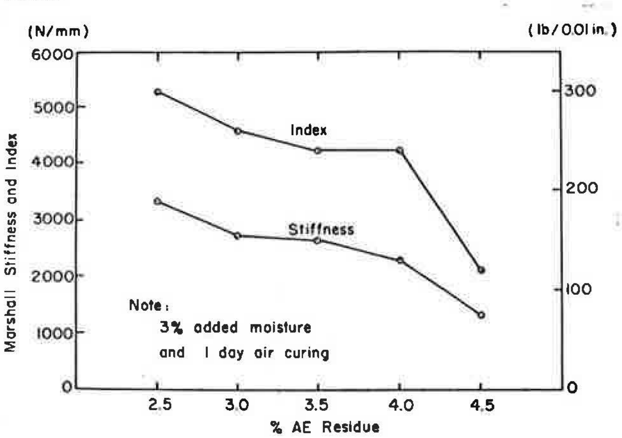


Figure 10. Effect of aggregate type, curing, and test temperature on tensile strength.

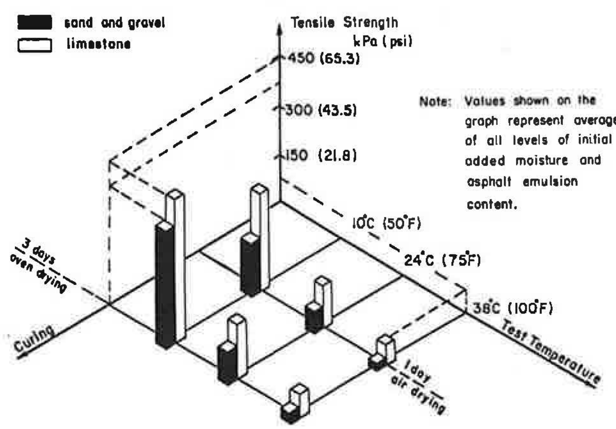
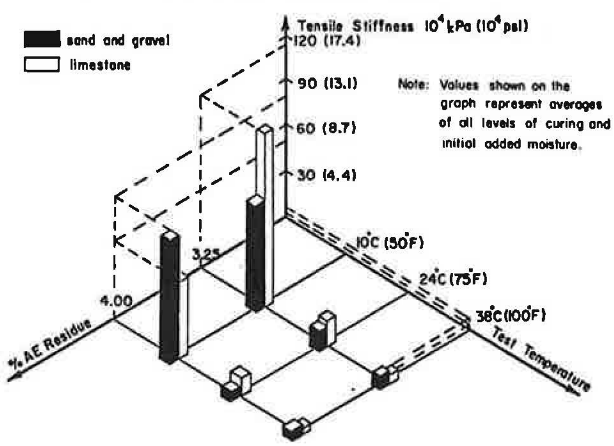


Figure 11. Tensile stiffness as a function of asphalt emulsion content, test temperature, and aggregate type.



temperature had an inverse effect on the stiffness value of the mixture. It may also be noted that AE content and aggregate type had a marked effect on tensile stiffness at a test temperature of 10°C (50°F).

Tensile strain at failure was affected by AE content and curing. Large tensile strains at failure were obtained for air-cured mixtures with 4 percent AE residue in comparison with oven-cured mixtures with 3.25 percent AE residue.

SUMMARY AND CONCLUSIONS

A laboratory technique for preparation of specimens to be used in the characterization of AE mixtures has been established. The technique was developed based on the coating of the aggregate, the workability of the mix, and the curing rate of the mixture before and after compaction. The mixture was characterized by using a modification of the Marshall method and the indirect tensile test. A modified water-sensitivity test developed by the Asphalt Institute was used to evaluate the resistance of an AE mixture to moisture.

The optimal initial added-moisture and AE contents should be selected to provide the best AE coating of aggregate particles. Two levels of added-moisture content and three levels of AE content would be adequate for the design of the mixture. Evaluating the mixture at two curing periods that represent the initial and long-term curing conditions would provide good understanding and control of mix performance.

The interaction of initial added-moisture and AE contents had a marked effect on the modified Marshall stability as well as Marshall stiffness and index of the mixture. There is an optimal liquid content that provides a mix with a maximum Marshall stability value. This liquid content was found to be in the range of 4-4.5 percent by weight of dry aggregate (for the materials and mixing procedures used in this study).

Test temperature and curing both had a substantial effect on the tensile properties of the AE mixtures. Moreover, the tensile strength of the mixture was markedly affected by aggregate type and initial added-moisture content. The AE content has a significant effect on tensile stiffness and tensile strain at failure.

The water-sensitivity test should play a major role in the evaluation of AE mixtures. Characterization of mixture specimens both before and after vacuum saturation would be beneficial in providing more realistic results, and this would in turn make it possible to establish better control over mixture properties.

ACKNOWLEDGMENT

The contents of this paper reflect our views, and we are responsible for the facts and the accuracy of the data presented.

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Use of the Hveem Stabilometer Test in Design Procedure for Emulsified-Asphalt Mix

LLOYD D. COYNE

The selection, proportioning, testing, and criteria recommended for the various uses of emulsified-asphalt mixes are discussed. Mix design incorporates the use of Hveem equipment to measure mix stability and cohesion. New testing techniques include the use of vacuum curing and vacuum saturation. The reasons for each test are reviewed. The mix-design procedure appears suitable for predicting the performance of emulsified-asphalt mixes. The intended use of the mix dictates the procedure and criteria to be used.

In the design of emulsified-asphalt mixes (EAMs), the intended use of the material determines the mix-design procedure and criteria. This paper discusses the design of such mixes according to their intended use.

CONSTRUCTION AID

Small percentages of emulsified asphalt (2-3 percent) may be added to sands and granular bases as part of the normal mixing water during the construction operation. The emulsified asphalt imparts cohesion to otherwise noncohesive materials, minimizing segregation during placement. It also aids in compaction and allows these materials to be used as a base and working table. The use of primes can frequently be eliminated. No testing of the EAM is required, provided the untreated aggregate meets the specifying agencies' requirements.

Table 1. Properties of aggregates suitable for base treatment with emulsified asphalts.

AASHTO or ASTM Test Method	Property	Processed Dense-Graded Aggregates[a]	Sands: Poorly Graded	Sands: Well-Graded	Sands: Silty	Semiprocessed Pit- or Bank-Run Aggregates: Low Sand	Semiprocessed Pit- or Bank-Run Aggregates: High Sand	Total
C136	Gradation (percentage passing)							
	1.5 in	100				100	100	
	1 in	90-100				80-100	80-100	
	0.75 in	65-90				-	-	
	0.5 in	30-60	100	100	100	-	-	
	No. 4		75-100	75-100	75-100	25-50	50-85	
	No. 16	13-30	-	35-75	-	10-30	30-75	
	No. 50	7-25	-	15-30	-	-	-	
	No. 100	5-18	-	-	15-65	-	-	
	No. 200	4-12	0-12	5-12	12-25	3-8	3-15	
C2419	Minimum sand equivalent (%)	35	35	35	35			35
T190	Minimum untreated resistance R-value	78	60	60	60			60
T97	Maximum loss (Los Angeles rattler at 500 revolutions)	50						50

[a] Must have at least 65 percent by weight crushed particles.

Table 2. Properties of processed, dense- and open-graded aggregate suitable for wearing surface with emulsified asphalts.

ASTM or Other Designation	Property	Dense-Graded Aggregate: A	B	C	D	E	F	Open-Graded Aggregate: Coarse[a]	Medium[b]	Fine[c]
C136	Gradation (percentage passing by weight)									
	1-1.5 in	100						100		
	1 in	90-100	100					95-100	100	
	0.75 in	-	90-100	100				-	90-100	-
	0.5 in	60-80	-	90-100	100			25-60	-	100
	0.375 in	-	60-80	-	90-100	100		-	20-55	85-100
	No. 4	25-60	35-65	45-70	60-80	80-100	100	0-10	0-10	-
	No. 8	15-45	20-50	25-55	35-65	65-100	95-100	0-5	0-5	0-10
	No. 16	-	-	-	-	40-80	85-100	-	-	0-5
	No. 30	-	-	-	-	20-65	70-95	-	-	-
	No. 50	3-18	3-20	5-20	6-25	7-40	45-75	-	-	-
	No. 100	-	-	-	-	3-20	20-40	-	-	-
	No. 200	1-7	2-8	2-9	2-10	2-10	9-20	0-2	0-2	0-2
D2419	Minimum sand equivalent (%)	45	45	45	45	45	45	-	-	-
C131	Maximum loss (Los Angeles rattler 500 revolutions)	40	40	40	40	-	-	40	40	40
California 205 E[d]	Minimum crushed faces (%)	65	65	65	65	65	65	65	65	65
C88	Maximum soundness (five cycles)	12	12	12	12	12	12	-	-	-
D3042	Maximum acid resistance[e]	10	10	10	10	10	10	10	10	10

[a] Thickness of course > 3 in.
[b] Thickness of course = 2-3 in.
[c] Thickness of course = 1-2 in.
[d] State of California test method.
[e] Applicable to limestone surface mixes.

TREATED BASE

Emulsified asphalt has been used to upgrade the quality of granular materials by providing tensile strength and reducing the movement of moisture into the mix. The design of emulsified-asphalt-treated bases includes the steps discussed below.

Selection of Aggregates

Aggregates suitable for emulsified-asphalt-treated bases consist primarily of granular materials: open-graded aggregate, processed dense-graded aggregate, natural gravel, slags, shells, reclaimed demolition waste, synthetic aggregate, sands, and silty sands. Clayey sand, loam, silty sands, and sandy clay may also be suitable. Heavy clays are best handled by using other stabilizing materials, such as lime or cement.

The sand-equivalent test is used to detect the presence of clay-type fines or sand in the aggregate. Predominantly granular materials with a sand equivalent greater than 35 percent can usually be stabilized by using emulsified asphalt. The chance for success with materials that have a lower sand equivalent depends on the nature of the fines and the ability of the asphalt to retard the movement of moisture into these fines. Materials that have a sand equivalent of less than 25 percent are usually not suitable for emulsified-asphalt mixes.

Aggregates found to be suitable for base treatment and wearing surface with emulsified asphalts are given in Tables 1 and 2.

Selection of Emulsified Asphalt

In selecting the type of emulsified asphalt, consideration must be given to aggregate type and gradation, climatic conditions, and construction methods. Three categories of mixes are suggested: sand mix, coarse-aggregate mix, and dense mix. The sand mix (i.e., CMS-2S) contains some solvent to allow it to coat dry or damp sands and intermediate-graded aggregates. Coarse mixes (i.e., CMS-2) are used with open-graded aggregates. The solvent in these emulsions assists in coalescence—i.e., the merging of asphalt particles into a tough asphalt film. The emulsifiers are selected to react rapidly with aggregate surfaces so as to resist rain damage and washoff. The dense mixes (i.e., CSS-1h) are designed for maximum mixing stability and are used with dense-graded aggregates that have a large surface area. They have a low viscosity that can be further reduced by adding water, thus facilitating longer workability times to ensure good coatings.

Specimen Preparation and Testing

The centrifuge kerosene equivalent (CKE) test is used for estimating the emulsified-asphalt content for dense-graded mixes. Trial mixes are normally made at emulsified-asphalt contents to 1.1, 1.4, and 1.7 times the CKE oil ratio. For open-graded mixes, the recommended emulsified-asphalt content, based on experience, is 4.5-6.5 percent, 5-7 percent, and 6-8 percent for coarse, medium, and fine gradation, respectively.

Mixing tests are conducted to determine the minimal water content required to obtain a workable, homogeneous mix. Generally, a 30-s cycle with a laboratory mechanical mixer or a 2-min bowl mix with a spoon is sufficient. A batch size of 1200 g is generally used. Smaller batch sizes may give misleading results.

Sand-mix and coarse-mix types of emulsified asphalt are mixed dry or at the aggregate in situ moisture content. Excess mixing water may induce stripping. Sufficient moisture is added to the dense-mix types of emulsified asphalt to just darken the aggregate (usually 3-5 percent). Coatings of 50 percent or more are acceptable; 100 percent is preferred, provided the increase in mixing water required to achieve 100 percent coating does not adversely affect compaction. The intermediate emulsified-asphalt content (1.4 x CKE) is generally selected for determining the optimal mixing-water content. Mixing-water contents for the other emulsified-asphalt contents are adjusted to give the same total water content (i.e., mixing water plus water in the emulsified asphalt).

In compaction, the mixes undergo 10-50 blows at 250 lbf/in^2 in the kneading compactor (ASTM D1561) and then a 40 000-lbf static load held for 1 min. The amount of fluids exuded during the static loading is noted. Mixes that exude 2-10 g of fluids are considered optimal for compaction. The moisture content in the mix should be adjusted to achieve this optimal fluid content. For open-graded mixes, it was necessary to reduce the static load to 20 000 lbf to avoid fracture of the aggregate.

The preliminary kneading compaction is felt to be desirable because it orients the aggregate particles and prevents fracture during subsequent static loading. Limited studies that incorporate the use of hand rodding appear to accomplish this objective so that it may be possible to eliminate the need for a costly kneading compactor. Densities achieved by using the modified procedure were found to correlate with field results.

Laboratory specimens are tested at two curing conditions, identified as "initial" and "final" cure. The initial cure consists of a 24-h ambient cure (73 ± 5°F) in the mold. The final cure consists of a total seven-day cure that includes three days in the mold followed by four days of room-temperature vacuum desiccation at 10-20 mm of Hg to facilitate the removal of moisture.

Early rainfall damage is of concern mainly in open-graded mixes, because of their high permeability. Dense mixes are generally not susceptible to early rainfall damage because of their lower permeability. The resistance of a particular mix to possible early rainfall damage is measured by a washoff test.

Water intrusion from surface or subsurface water can damage some mixes. Dense-graded mixes are subjected to vacuum saturation to evaluate the effect of water. After the initial cure, the specimen is covered with water, and the desiccator is evacuated to 100 mm of Hg for 1 h. Then the vacuum is slowly released, and the specimen is allowed to soak for an additional hour.

Strength tests are then run to determine whether the EAM will meet minimal bearing-strength requirements when it is saturated with water. Open-graded mixes are not tested for strength. These mixes are considered to have adequate stability. The Hveem stabilometer (R-value) and cohesiometer (C-value) tests are run on dense-graded mixes after the initial- and final-cure vacuum-saturated condition. Mixes are tested at 73 ± 5°F to simulate temperatures within the treated base. The results are expressed as an R_t value (R + 0.05C).

WEARING SURFACE

Both dense- and open-graded EAMs have been used as the wearing surface. The highest emulsified-asphalt content consistent with mixing and stability requirements is recommended. Typically, 6-15 percent emulsified asphalt is used depending on the aggregate selected.

Procedures for the selection of materials and sample preparation are similar to those recommended for treated bases. Hveem stabilometer and cohesiometer tests are run after the final cure. Specimens are tested at 140° ± 5°F.

SUMMARY

A summary of the design criteria for base and wearing surfaces is given in Table 2 in the paper by Waller elsewhere in this Record. In addition to the requirements given, the mix must be reasonably workable—i.e., not too stiff or sloppy. The intended use of the material dictates the design procedures followed, as summarized below:

1. Construction aid—No testing is required, provided the untreated aggregate meets the requirements of the specifying agency.
2. Treated base—The material must meet minimum coating and stability requirements. The resistance R_t-value is used to measure mix stability after the initial-cure and final-cure vacuum-saturated condition.
3. Wearing surface—Mixes are tested at 140°F for both stability and cohesion by using the Hveem procedures.

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Mix-Design Procedures for Open-Graded Emulsion Mixes

R. G. HICKS, JEAN WALTER, AND RONALD WILLIAMSON

Three mix-design procedures for open-graded asphalt emulsion mixes (Chevron USA, Inc.; U.S. Forest Service Region 6; and Federal Highway Administration Region 10) are summarized. The basic steps used in each method and the major differences among the methods are discussed. Finally, the results of a limited round-robin test program are presented that indicate very little difference in the recommended emulsion and water contents for the three methods. All three procedures are reasonably simple and require little time when compared with those for dense-graded emulsion mixes. However, there is still a need to adopt a standard method of evaluating the strength (or stiffness) of the mix to obtain layer coefficients. Currently, the diametral modulus appears to be the most promising test method available.

Open-graded emulsion mixes are mixtures of open-graded aggregate and emulsified asphalt, generally CMS-2. An open-graded mixture is characterized by a high void content, on the order of 20-30 percent, and less than 10 percent of the material passing the 2-mm (No. 10) screen. The mixtures are cold mixed in a pugmill and then placed and compacted on the roadway by using conventional equipment and construction procedures. Open-graded mixtures possess high permeability, which aids in curing, facilitates the removal of ground and surface water that can weaken the structural section, and minimizes the chances of hydroplaning. Open-graded mixes also appear to be more flexible than dense-graded mixes, causing an increase in resistance to fatigue and reflection-type cracking (1,2).

Open-graded emulsion mixes have been used extensively since 1965 in the Northwest in the construction of low-volume county and Forest Service roads that carry gross loads of as much as 890 kN (200 000 lbf). These mixes have several advantages over hot mix. They generally require less energy to produce, reduce air pollution, and do not create a fire hazard. They are often more economical than a hot mix, especially when long haul distances are required. In recent years, however, there have been reports of wide variations in the performance of open-graded mixes. Early failures of some projects have resulted in a reduction in use of open-graded emulsion mixes by one of its largest users, Region 6 of the U.S. Forest Service (2). Factors that contribute to reported failures include (a) poor (or no) mix design, (b) poor construction practices and production control, and (c) inadequate structural design.

This paper provides a summary of mix-design procedures for open-graded emulsion mixes used by three sources: Chevron USA, Inc., Region 6 of the U.S. Forest Service, and Region 10 of the Federal Highway Administration (FHWA). The steps used in each method to determine emulsion and water contents are presented, and the differences in the methods are discussed. Finally, the results of a limited "round-robin" study are presented to show the differences in the emulsion and water contents obtained by each method.

STEPS IN MIX DESIGN

In all emulsion-mix-design procedures, the ultimate objective is to determine the type and amount of emulsion needed to make a satisfactory mix. To do this, one must undertake the steps outlined below. If all of the steps are carried out, the chances of a successful project are increased considerably.

Select an Appropriate Aggregate

The gradation of the aggregate in open-graded mixtures is very important. Based on construction experience to date (1), no more than 10 percent passing the 2-mm (No. 10) sieve or 2 percent passing the 0.074-mm (No. 200) sieve is recommended. For open-graded mixes, a large amount of air voids in the compacted mix (about 20-30 percent) is necessary. A large percentage of fine aggregate and/or surface area can cause early breaking of the emulsion, result in improper coating and poor mix workability at laydown, and possibly slow the rate of strength development.

Another important factor in the success of open-graded emulsion mixes is the inherent stability of the aggregate, which depends on particle interlock as well as hardness and durability. Crushed aggregates are recommended. The recommended minimum amount of fractured faces is 60-75 percent (3).

Select the Type of Emulsion

In selecting the type of emulsion, consideration must be given to the aggregate type and gradation and climatic conditions. Medium-set emulsions are normally used for open-graded emulsion mixes. They are designed to break during mixing or shortly thereafter and allow compacting operations to be completed without any difficulty.

Medium-set emulsions can be anionic or cationic, depending on the electrical properties of the asphalt particle surface. If the emulsifiers used provide a negative surface charge, the emulsion is anionic. A positive surface charge occurs in cationic emulsions. Cationic emulsions can be used with almost all types of aggregates because practically all aggregate types possess a negative surface charge. Cationic emulsions extend the paving season since they can be used in cooler weather. In addition, since they will break without evaporation, they are not as easily damaged by sudden showers, which means that they will not wash away quite as readily (4).

The choice between cationic and anionic grades depends on the aggregate as well as on the mix coating, curing, and adhesion characteristics. The viscosity of an emulsion increases with increasing asphalt residue content and with decreasing penetration grade. High-viscosity emulsions (CMS-2 and CMS-2h) are preferred for use in open-graded mixes to minimize runoff and to provide greater film thickness and durability.

Select the Amount of Water and Emulsion

The procedures used to select emulsion and water contents differ among agencies; all agencies, however, prepare trial mixtures, and the one selected is usually the one that gives the most satisfactory results in terms of workability, coating percentage, and film thickness.

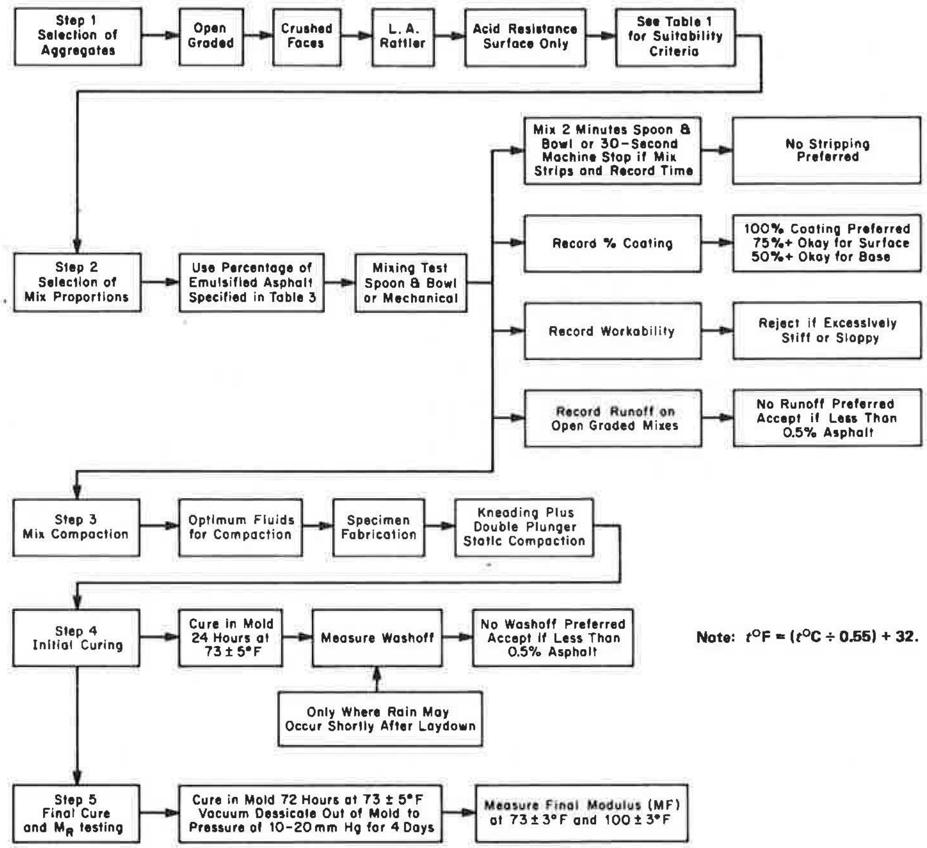
Compact the Mix

Laboratory specimens are normally fabricated at optimum fluid content (asphalt plus water), by first using a kneading compactor and then a double-plunger static load. Emulsified asphalt mixes must be compacted in the field to at least 95 percent of laboratory density to provide necessary aggregate interlock. Proper compaction is important if the mix is to achieve its design strength or stiffness.

Cure the Specimen and/or Subject It to Moisture

Under favorable curing conditions, properly designed emulsified-asphalt mixes can be opened to heavy traffic within 45 min to 1 h or to controlled traffic immediately. Rain damage is of concern in open-graded mixes during and shortly after construction. Open-graded mixes that

Figure 1. Testing schedule for emulsified-asphalt mixes: Chevron procedure.



incorporate CMS-2 and CMS-2h emulsion have been placed during light rain without difficulty, but in heavy rain the emulsion may wash off. The mix is susceptible to washoff until the emulsion breaks. The resistance of a particular mix to possible early damage from rainfall is measured by the washoff test in the Chevron procedure.

Test the Specimen

Various standard strength tests are carried out to ensure a stable mix and to select the appropriate thickness so that the mix will not rut or crack under load. The test most commonly used to evaluate this property is the diametral modulus (5).

CHEVRON PROCEDURE

Figure 1 shows the steps involved in using the Chevron procedure to test emulsified-asphalt mixes (6).

Selection of Aggregate

The types of aggregates considered suitable for open-graded emulsified-asphalt treatment include blast-furnace slag, coral, volcanic cinder, gravel, ore tailings, crushed ledge stone or rock, reclaimed aggregates, or other inert material. These aggregates must meet the requirements given in Table 1.

Selection of Emulsified Asphalt

Both anionic and cationic emulsified asphalts can be used. Medium-set emulsions are preferred for open-graded mixtures. However, other factors, such as job climatic conditions, the availability of water, and of course the availability of the product, influence the choice. The table below provides guidelines for selection of medium-setting emulsified asphalts other than CMS-2 and CMS-2h (note that CMS-2s grade is not used with open-graded mixes):

Grade Designation	Use as Aggregate	Rain Resistance	Construction Method
CMS 2s	Dry or damp low-sand-content gravels; well-graded or silty sands	Resistant to early rainfall	Travel plant or in-place mixing
MS 1 and MS 2	Dry or damp processed open-graded aggregates	Resistant to early rainfall	Central mix or travel plant

Selection of Mix Proportion

For open-graded mixes, trial mixes are prepared by using various emulsion contents. The batch weight used is a function of the maximum aggregate size, as given below, and should conform to the gradation specified (1 mm = 0.039 in; 1 kg = 2.2 lb):

Nominal Maximum Particle Size (mm)	Minimum Batch Weight (kg)
19.00	1.20
12.50	0.75
4.76	0.50

The emulsion content is selected from those given below:

Open-Graded Aggregate	Approximate Emulsified-Asphalt Content (percentage by weight of aggregate)
Coarse	4.5-6.5
Medium	5.0-7.0
Fine	6.0-8.0

Table 1. Open-graded aggregates suitable for emulsified-asphalt mixes.

ASTM Designation	Category	Aggregate Coarse[a]	Medium[b]	Fine[c]
C136	Gradation (percentage passing by weight)			
	38.1 mm	100		
	25.4 mm	95-100	100	
	19 mm	-	90-100	
	12.7 mm	25-60	-	100
	9.5 mm	-	20-55	85-100
	4.75 mm	0-10	0-10	-
	2.36 mm	0-5	0-5	0-10
	1.18 mm	-	-	0-5
	0.595 mm	-	-	-
	0.297 mm	-	-	-
	0.149 mm	-	-	-
	0.075 mm	0-2	0-2	0-2
D2419	Sand equivalent (%)	-	-	-
C131	Maximum loss (Los Angeles rattler at 500 revolutions)	40	40	40
California 205-E[d]	Minimum crushed faces (%)	65	65	65
C88	Soundness (five cycles)	-	-	-
D3042	Maximum acid resistance[e]	10	10	10

Note: 1 mm = 0.039 in.

[a]Thickness of coarse aggregate = 76 mm.
[b]Thickness of coarse aggregate = 51-76 mm.
[c]Thickness of coarse aggregate = 25-51 mm.
[d]State of California test method.
[e]Applicable to limestone surface mixes.

It is then mixed with aggregate that contains a minimal amount of water. (With porous aggregates, the emulsified-asphalt content should be increased by a factor of approximately 1.2. Porous aggregates are those that absorb more than 2 percent water by dry weight when tested by using ASTM C127.) The liquid materials are thoroughly mixed, and the workability is noted. The process is repeated by increasing the emulsion and/or water content until satisfactory results are obtained.

Upon completion, the mix is poured onto a 0.841-mm (No. 20) wire-mesh screen funnel and allowed to drain for 30 min. The runoff is collected in a tared can under the funnel, placed in a 110° ± 5°C (230° ± 9°F) oven, and dried to a constant weight. The percentage of runoff is computed as (final weight - tared weight)/batch aggregate weight x 100. The sample is removed from the screen funnel, and the emulsion content is selected by considering the following:

1. Coating—The coating is evaluated by visually examining the air-dried samples;
2. Workability—The mixing operation should be easy to perform;
3. Runoff—No runoff is preferred, but less than 0.5 percent is acceptable; and
4. Job conditions—Such factors as the availability of water, weather conditions, and the mixing process also influence the selection of the type and grade of emulsified asphalt.

Specimen Fabrication

The mix specimen is fabricated at optimum emulsion and moisture content by using a compactive effort similar to that obtainable in the field. Enough mix is poured into a mold to form a specimen that is 101.6 mm (4 in) in diameter by 63.5 mm (2.5 in) high. Preliminary compaction is accomplished by applying approximately 20 tampings at 1723.5 kPa (250 lbf/in²). Finally, a 178-kN (40 000-lbf) static load is applied by using the double-plunger method. This is currently being changed to 89 kN (20 000 lbf) to minimize aggregate degradation.

Initial Curing and Washoff Test

The washoff test measures the ability of an open-graded mix to withstand rain damage. This test is required only if rain is anticipated shortly after laydown.

The compacted sample, still in the mold, is first cured at a temperature of 22° ± 3°C (73° ± 5°F) for 24 h and then placed on a pedestal and wire screen. About 200 cm² of water is poured over the sample, and the washoff is collected in a tared container that is placed under the screen. The mix is allowed to drain for 30 min. The container is dried to a constant weight at a temperature of 110° ± 5°C (230° ± 9°F), and the weight of the residual asphalt washoff is determined as follows: Percentage of washoff = (weight of residual asphalt washoff/weight of specimen aggregate) x 100.

Final Cure and Diametral Modulus Testing

A different sample is cured in a mold for 72 h at 22° ± 3°C (73° ± 5°F) and then vacuum desiccated out of the mold for four days at 10-20 mm (0.4 to 0.8 in) of Hg. The diametral resilient modulus (M_R) test is used to determine the structural contribution of the mix in the pavement (5). The test is performed at 22° and 38°C (73° and 100°F) by using a 0.1-s pulse load applied every 3 s across the diameter of the test specimen. For open mixes, the test is normally performed by using a confining pressure. The horizontal deflection to the applied load is measured by a pair of transducers mounted in a yoke that is clamped to the specimen. M_R is calculated as follows:

$$M_R = [P(\mu + 0.2734)/t\Delta] = 0.6234(P/t\Delta) \quad (1)$$

where

P = dynamic load (lbf),
μ = Poisson's ratio (normally assumed to be 0.35),
t = thickness of the specimen (in), and
Δ = horizontal deflection (in).

The results of this step are not currently used in selecting mix proportions.

U.S. FOREST SERVICE PROCEDURE

Figure 2 summarizes the design procedure for open-graded emulsion mixes used by Region 6 of the U.S. Forest Service (7). The individual steps are described below.

Selection of Aggregate

The aggregate gradation suggested by the Forest Service (7) is given below (1 mm = 0.039 in):

Sieve Size (mm)	Percentage Passing
25.4	100
12.5	45-70
4.75	0-20
2.0	0-6
0.425	
0.075	0-2

The following weights (8) are suggested for the air-dry samples (1 kg = 2.2 lb):

Nominal Maximum Size of Particle (mm)	Minimum Weight of Sample (kg)
9.5	1
12.5	1
19.0	2
25.4	5
38.0	10

The samples are then tested according to AASHTO T11 and T27, and the results are compared with the specified gradation.

Selection of Emulsion Type

Asphalt emulsion grades CMS-2, CMS-2h, and CMS-2s are normally used for open-graded mixes by the Forest Service. Aggregate characteristics, particularly gradation, are the primary controlling influence on the selection of the type of emulsified asphalt. Grades CMS-2 and CMS-2h are used when aggregate passing the 2-mm (No. 10) sieve is less than 6 percent and that passing the 0.074-mm (No. 200) sieve is less than 2 percent. CMS-2s is used when the surface area increases because of an increase in the percentage of aggregate passing the 2-mm and 0.074-mm sieves.

Selection of Mix Proportion

The emulsion content is determined from the oil-ratio (K_c) test. The following formula is used to estimate the percentage starting content: Beginning emulsion content = (K_c x 1.5) + 3.5. If the oil-ratio equipment is not available, the following table can be used to determine the beginning emulsion content:

Aggregate Absorption (%)	Approximate Emulsion Content (percentage by weight of aggregate)
<1	5.0
1-2	6.0
>2	7.0

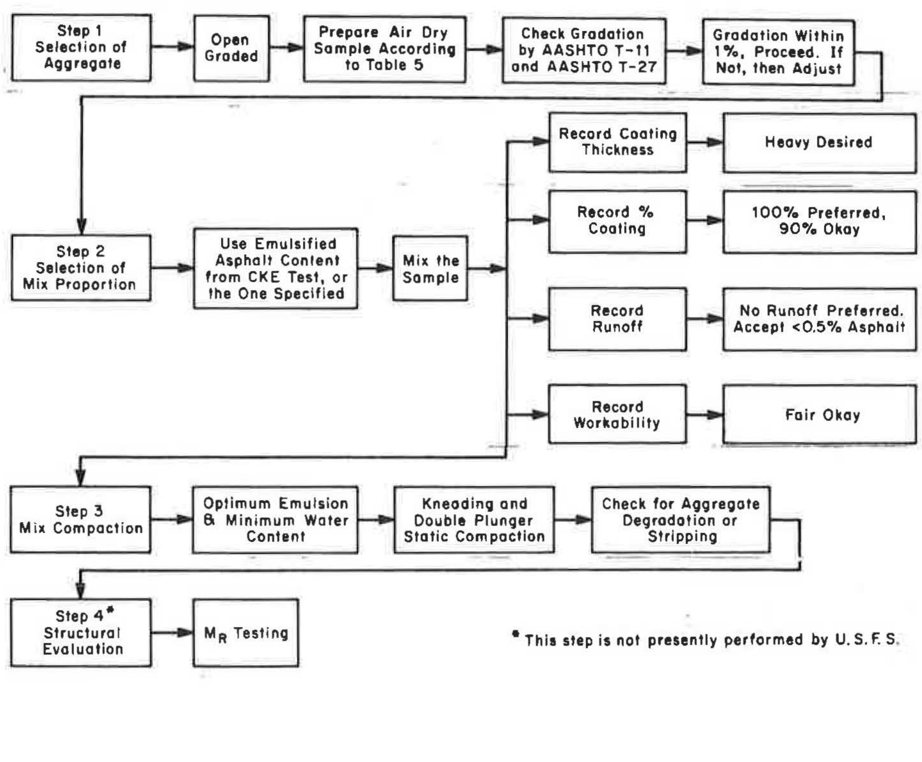
Once the beginning emulsion content is estimated, a number of trial mixes are prepared. The percentage of water and asphalt is varied in each case, and the coating thickness, percentage of coated area, excess fluids, and workability are recorded. The emulsion content and lower and upper limits of the water content are determined as follows:

1. For the minimum acceptable mix, 90 percent of the area is coated by emulsion and the workability is fair. The optimum emulsion content is reached when coating is heavy and the coated area is 100 percent. These evaluations are done visually.
2. The lower limit for moisture content is the hygroscopic moisture of the aggregate. The upper limit is the highest water content at which no appreciable excess fluids are present.

Specimen Fabrication

A 1200-g (2.65-lb) mixture is fabricated at the lower limit of moisture and optimum emulsion content in a mixing bowl. Half of the mix is then placed in a 101.6-mm (4-in) diameter compaction mold and rodded 20 times in the center and 20 times around the edge with a bullet-nosed steel rod. The second half is then added, rodded, and compacted by using a kneading compactor. Approximately 20 tamping blows are applied at 1.7 MPa (250 lbf/in^2) to accomplish preliminary compaction. Final compaction is done by applying 150 tamping blows at 3.4 MPa (500 lbf/in^2) and then applying a leveling load of 56 kN (12 600 lbf) by using the double-plunger method. In the case of unstable material, a static load of 178 kN (40 000 lbf) is applied by using the double-plunger method. Two specimens

Figure 2. Testing schedule for open-graded mixes: U.S. Forest Service procedure.



are prepared, and the dry density is determined after the samples have been oven cured for 24 h at 110° ± 5°C (230° ± 9°F). This value is used only for field compaction control.

Strength Test

No standard strength tests have been adopted by the Forest Service. However, some stable cores have been tested for resilient modulus, from which strength coefficients have been obtained by using the AASHTO Interim Guide (7).

FHWA PROCEDURE

Figure 3 shows the steps involved in the design procedure for open-graded asphalt emulsion mixes used by Region 10 of FHWA. Each of these steps is described below.

Steps Currently in Use

The typical gradations used by FHWA are given in Table 2. Gradings A and B are normally recommended for base courses, whereas grading C is recommended for surface courses. All aggregates should be free of clay, loam, and vegetable matter. The aggregate selected should also meet the quality requirements given. The gradation is checked by carrying out AASHTO tests T27 and T11.

Selection of Emulsified Asphalt

Only cationic asphalt emulsions, grade CMS-2 or CMS-2h, are recommended.

Selection of Mix Proportion

Once it has been determined that the aggregate meets the requirements given above, an oil-ratio test is carried out to determine the beginning emulsion content. The relation used to estimate beginning emulsion content is identical to that used by the U.S. Forest Service.

About 15-20 samples, weighing 500 g (1.10 lb) each, are prepared by thoroughly mixing the emulsion and aggregate with a putty knife. The mixing process begins by adding the beginning emulsion content and testing for harshness. A mix that shows stiffness in the first 15-20 s of mixing is harsh.

Figure 3. Testing schedule for emulsified-asphalt mixes: FHWA procedure.

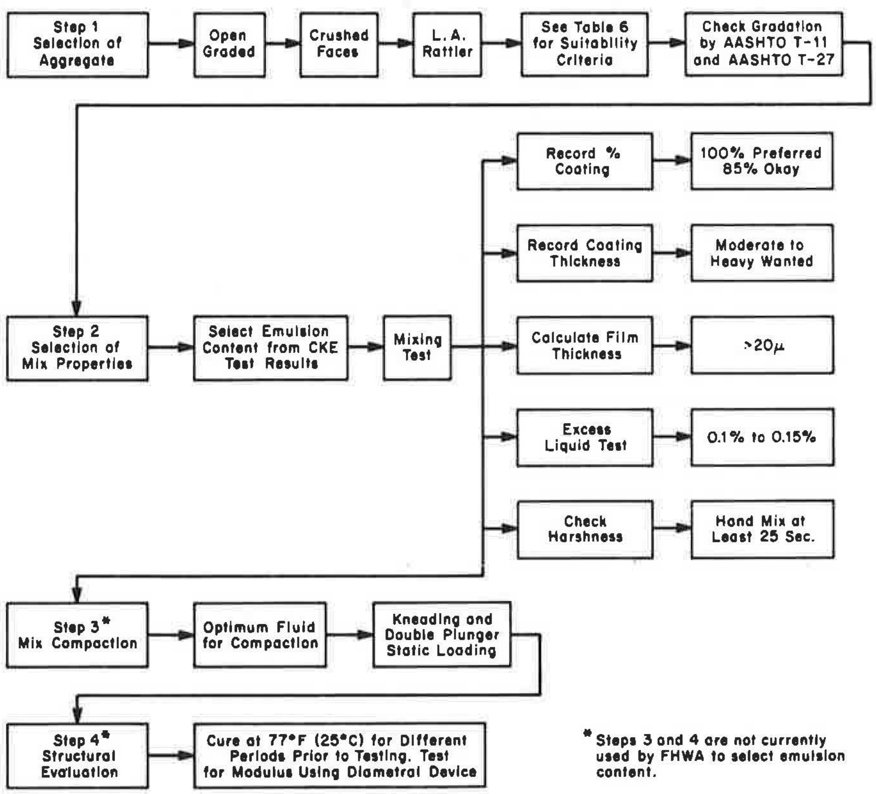


Table 2. Aggregates suitable for emulsified-asphalt mixes: FHWA procedure.

AASHTO Designation	Property	Grading A	Grading B	Grading C
M-92	Gradation (percentage passing)			
	38.0 mm	100		
	24.4 mm		100	
	19.0 mm			100
	2.0 mm	0-7	0-7	0-7
	0.075 mm	0-2	0-2	0-2
T-176	Minimum sand equivalent (%)	45	45	45
T-96	Maximum loss (Los Angeles rattler at 500 revolutions)	35	35	35
California 205-E	Minimum crushed faces (%)	70	70	70
T-104	Sodium sulfate soundness loss on portion retained on 4.75-mm sieve (%)	12	12	12

Acceptable mixes do not become stiff until 30-45 s of mixing.

An acceptable emulsion content is determined by evaluating mix characteristics that must fall within an acceptable range, as follows:

1. Thickness of coating—The coating, which is evaluated visually, must be moderate to heavy.
2. Percentage of coating—The percentage of coating is also evaluated visually and should be within 90-100 percent. The absolute lower limit is 85 percent.
3. Film thickness—Film thickness (in micrometers) is calculated by using the following formula: Film thickness = (48.7 x percentage effective asphalt)/surface area (from centrifuge kerosene equivalent). This should be 20 μm or more where the effective asphalt content is the percentage residual asphalt in the emulsion minus the percentage asphalt retained in the tared pans.
4. Excess liquids—Water is added before emulsion, starting with 1.0 percent and increasing by 1.0 percent increments. The process is continued until a measurable amount of excess liquid can be poured out of the mixture when the beginning emulsion content is used. The amount of excess liquid should be slight: 0.1-0.15 percent.
5. Harshness—The mix should be easy to mix by hand for at least 25 s.

Proposed Steps

The steps described below are proposed, but not currently used, by FHWA to select emulsion or water content.

Specimen Fabrication

The procedure used to fabricate a specimen is similar to that used by the Forest Service; that is, specimens 63.5 mm (2.5 in) in height by 101.6 mm (4 in) in diameter are compacted by using the kneading compactor and double-plunger static compaction. Dry density and free fluids (if any) are recorded.

Mix Cure

The rate at which open-graded emulsions develop strength is determined by testing the samples at 25°C (77°F) for various periods of cure.

Strength or Stiffness Test

A diametral test (5) is being considered for use in evaluating open-graded emulsified-asphalt mixes, but no strength test is actually included in the FHWA procedure.

DIFFERENCES IN MIX PROCEDURES

Table 3 summarizes the three mix-design procedures described above. Differences in the procedures are discussed below.

Mix Evaluation

Coating is determined by observing the emulsion mix after thorough mixing and drying. This is a subjective evaluation, and the results are recorded as a numerical percentage. Each of the agencies discussed above has various recommended coating requirements, ranging from 75 to 90 percent minimum. Factors that affect the degree of coating include the quantity of fines in the aggregate, the mixing effort, the charge on the aggregate surface, moisture, mixing time, emulsion type, and emulsion and water contents.

Curing

Three methods have been used to cure emulsion mixes: air, oven, and vacuum curing.

Air curing has been used most extensively, but its effectiveness is directly dependent on the height of the sample. As the height increases, the time required to reach final curing increases. This can create considerable delay. Currently, two methods are used to achieve curing in the

Table 3. Differences between mix-design procedures for open-graded emulsion mixes.

Design Step	Chevron and Asphalt Institute	U.S. Forest Service	FHWA
Evaluation of aggregate			
Gradation	Three proposed gradations	One gradation	Three possible gradations
Quality	Acid resistance, surface only[a]; Los Angeles rattler; crushed faces	Covered in specifications	Sand equivalent, Los Angeles rattler, sodium sulfate soundness
Type of emulsion	CMS or MS; Chevron selection based on aggregate, rain, and construction conditions	CMS-2 and CMS-2h or CMS-2s, depending on gradation	CMS-2 and CMS-2h
Mix proportion			
Beginning asphalt content	From table relating emulsion content and gradation	CKE test: 1.5 K_c + 3.5 or from table content and absorption	CKE test: 1.5 K_c + 3.5
Mixing-moisture content	Minimum to achieve coating or in situ moisture content	When aggregate darkens	Natural aggregate moisture
Criteria for optimum fluid content	Coating, thickness, workability, percentage runoff	Coating, workability, thickness, percentage coated area, little or no excess fluids	Workability, coating thickness, percentage coated area
Mixing time	2 min hand mixing or 30 s by machine	30 ± 5 s hand mixing	Effective asphalt content at 30-45 s hand mixing
Specimen fabrication			
Compaction	Kneading and double-plunger static loading compaction	Kneading and double-plunger static loading compaction	-
Curing	In mold 24 h at 22°C for washoff test; in mold 72 h at 22° ± 3°C; in vacuum of 10-20 mm Hg four days for M_R test	In oven at 110° ± 5°C for 24 h	
Measurements and calculations	Dry density	Dry density	-
	Mixing-fluids content must be adjusted to minimize fluid exudation	Effective emulsion content	
Mixture testing	Washoff test on air-cured samples Resilient modulus test on air- and vacuum-cured samples	-	-
Evaluation	75-100 percent coating, less than 0.5 percent runoff, good workability, less than 0.5 percent washoff, resilient modulus	Moderate to heavy coating, 90-100 percent coated area, no excess fluids, fair workability	90-100 percent coating, moderate to heavy coating, 20-μm minimum film thickness, 0.1-0.15 percent excess liquids, good workability

Note: t°C = (t°F - 32)/1.8.

[a] Chevron only.

air: (a) leaving the sample to cure in the mold and (b) removing the sample from the mold and placing it in a rubber membrane to cure. Open-graded emulsion mixes may slump unconfined without any preliminary cure.

Oven curing is used by some agencies, but additional research is needed to obtain a suitable oven temperature and duration period. Temperatures above 38°C (100°F) can result in loss and/or redistribution of the residual asphalt, and the sample may slump and fall apart.

Vacuum curing is used by Chevron USA, Inc., for dense mixes and open mixes. A sample fabricated to 63.5 mm (2.5 in) in height by 101.6 mm (4 in) in diameter is initially cured by placing the mold in a horizontal position for 72 h at a temperature of 22° ± 3°C (73° ± 5°F). The specimen is then removed from the mold and vacuum desiccated for four days to achieve the ultimate cured condition.

Compaction

Several methods have been used to compact emulsion mixes.

The most widely used is the kneading compactor. The compaction equipment and the procedure are described in detail in ASTM D1561-65. Test specimens 63.5 mm (2.5 in) in height by 101.6 mm (4 in) in diameter are prepared. To prevent degradation from occurring during compaction, a protective rubber pad is often used (7,9,10).

The Marshall hammer has been used in research at Oregon State University (OSU). The results of the OSU investigations indicate that, in good-quality aggregates, degradation will not take place. The vibratory compactor has also been used at OSU. A preliminary evaluation of the various techniques would suggest that the vibratory hammer produces a sample with the least amount of degradation. Studies completed at OSU (1) also indicate that there are differences in the rate of stiffness development for mixes prepared with Marshall and vibratory techniques.

Design Emulsion Contents

A limited round-robin study has been performed by using the three mix-design procedures discussed in this paper. Table 4 summarizes the results for three crushed basaltic aggregates (Forest Service gradation as given on page 28). The recommended amounts of emulsion are quite consistent, but the water content tends to vary from one agency to another. This could possibly be caused by differences in absorbed moisture at the time of testing or differences in the mix-design procedures.

DISCUSSION OF OPEN-GRADED EMULSION MIXES

Open-graded emulsion mixes have been and will continue to be used. They offer considerable advantages and yet, because of the newness of the materials, their use has also posed some problems. Some of their benefits and limitations are discussed briefly below.

Benefits

Some of the benefits of open-graded emulsified-asphalt pavements are as follows:

1. Resistance to cracking—Experience has proved these pavements to be highly resistant to cracking even under heavy loads. These pavements have successfully carried logging trucks with gross loads as heavy as 890 kN (200 000 lbf) without distress.
2. Reduced pollution—Dust pollution at the mixing plant is minimized, and air pollution associated with aggregate drying is eliminated.
3. Reduced asphalt oxidation—Because the aggregate and emulsion are mixed cold, severe asphalt oxidation in the mixing process does not occur. The thicker asphalt films also tend to retard oxidation of the asphalt in the pavement.
4. Lower costs—Construction costs tend to be lower because fewer pieces of construction equipment are needed. Because the mixtures are prepared cold, no dryer is needed and there are savings in fuel costs.
5. Energy—With today's awareness of the need to save and conserve energy in all of its forms and the need for more efficient use of the energy we must consume, this construction procedure offers many benefits. In addition to the monetary savings that can be expected, there is also the important consideration of the potential total savings of energy. Substantial amounts of energy can be saved in heating and drying aggregates and heating and maintaining the temperature of liquid asphalt by using open-graded emulsified-asphalt mixtures.

Limitations

The many limitations on the wide use of emulsion mixes can be reduced to three essential points:

1. Emulsion mixes may never have the same versatility with respect to climatic conditions that hot mixes have. However, this should not in itself be a reason for avoiding emulsion mixes.
2. There is no nationally accepted standard laboratory procedure for the design of emulsion mixes. Open-graded emulsion-mix design is based mainly on the spoon and bowl mixing test, and each agency has its own standards that the mix must meet to be acceptable from the point of view of workability, coating thickness, and percentage of aggregate coated. Even if runoff, washoff, and compaction tests are specified to minimize loss of emulsion and to maximize density, no standard strength test is available to the design engineer who faces problems with pavement thickness design. More data on the structural performance of emulsion-mix pavements are needed to determine a standard strength test that reflects the actual behavior of emulsion-mix pavements.
3. Design and construction experience with emulsion mixes is lacking. To overcome this problem, we suggest a preconstruction conference among contractors, agencies, and materials suppliers and the development of a manual of construction practices.

SUMMARY

Although there is no standard mix-design procedure for open-graded emulsion mixes, the methods described in this paper are encouraging. Open-graded emulsified-asphalt pavements designed by these methods have provided enough initial strength to support heavy vehicle loads immediately after construction and are durable enough to perform as well as a hot mix.

The following points have been noted in this study:

1. Mix designs adopted by different agencies are

Table 4. Results of round-robin study of mix-design procedures.

Aggregate	Agency	Emulsion Type	Emulsion Content (%)	Water Content (%)
Rivergate basalt	Forest Service	CMS-2	6.7	2.5
	FHWA	CMS-2	7.0	<2.0
	Chevron	CMS-2	5.0-7.0	<2.0
Berry Creek	Forest Service	CMS-2	7.0-7.7	<3.0
	FHWA	CMS-2	6.0	<3.5
	Chevron	CMS-2	5.0	2.0
	OSU (Forest Service method)	CMS-2	6.0	<4.0
Eckman Creek	Forest Service	CMS-2	5.5-6.0	5.5-6.0
	FHWA	CMS-2	7.0	<8.0
	Chevron	CMS-2	6.0	2.0
	OSU (Forest Service method)	CMS-2	6.0	4.0

essentially a trial-and-error process, since a universally acceptable design procedure is still not available.

2. Curing is not as important a consideration in open-graded mixes as in dense mixes, although it will be slower in the cooler climates.

3. Aggregates used in open-graded mixes must be clean, of good quality, uniform in size, and rough in texture. Dirty aggregate causes coating problems, and unsound aggregate causes performance problems.

4. Open-graded emulsion mixes, though they possess little if any unconfined strength, can support gross loads as heavy as 890 kN (200 000 lbf) without significant rutting.

5. Open-graded mixes should not be placed in heavy rain, for this may result in washoff of emulsion from the aggregate. The emulsion must break to be safe from washoff.

Although a number of designs for open-graded emulsion mixes are available, the results in terms of design emulsion and water content are similar. It would be desirable to establish one of the methods discussed in this paper as a standard for others to use.

However, design strength criteria are badly needed. Values used to date have been chosen based on limited experience and engineering judgment. A means of better predicting the effects of curing is needed in the design process. Whatever is done should ensure that laboratory curing conditions compare as much as possible with curing conditions in the field.

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Mechanistic Thickness-Design Procedure for Soil-Lime Layers

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A mechanistic thickness-design procedure for soil-lime pavement layers is presented. The procedure, which is based on a stress-dependent finite-element computer model called ILLI-PAVE, is limited to pavements constructed of a cured soil-lime layer and a nonstructural surface course (surface treatment or thin asphalt concrete). Design input data are soil-lime strength and modulus, subgrade resilient modulus, and estimated traffic. The procedure assumes that the soil-lime mixture is capable of developing significant increase in strength (relative to the strength of the natural soil) and that quality field construction and control are achieved.

Thompson (1) has considered the use of soil-lime mixtures for the construction of low-volume roads. The topics reviewed by Thompson include mixture properties and characteristics, soil-lime pavement layers (load-deflection behavior and field performance) and thickness-design concepts. Thompson concluded that "a more mechanistic and rational design procedure would be appropriate for designing pavements containing lime-treated soils" (1).

A repeated-loading study of soil-lime layers by Suddath and Thompson (2) indicated that a stress-dependent finite-element computer model called ILLI-PAVE adequately predicted load-deflection behavior. The study demonstrated that high load-carrying capacities can be developed by a soil-lime structural paving layer.

A simple thickness-design procedure for soil-lime layers has been developed based on the ILLI-PAVE model. The procedure is limited to pavements constructed of a cured soil-lime layer and a nonstructural surface course (surface treatment or thin asphalt concrete).

DEVELOPMENT OF DESIGN PROCEDURE

General Observations

Normally, only soil-lime mixtures that develop significant strength increases—mixtures that Thompson has called "lime reactive" (3)—are used in constructing structural paving layers. These mixtures have compressive and shear strength, and the factor that controls layer thickness is the flexural stress at the bottom of the soil-lime layer. The design procedure is based on the concept of a limiting stress ratio (S = flexural stress/flexural strength), which accounts for mixture fatigue behavior.

Description of ILLI-PAVE

ILLI-PAVE, a stress-dependent finite-element computer program, is described elsewhere (4). The program was developed based on the early work of Duncan, Monismith, and Wilson (5). Several University of Illinois modifications concerning input-output routines and material-failure models have been incorporated into ILLI-PAVE.

Material Properties

Soil-lime mixtures were considered to be linear-elastic materials. Cured reactive soil-lime mixtures develop significant flexural strengths and flexural moduli of elasticity (1,6). Typical ranges of flexural moduli are given below (compressive strength = 4 x flexural strength; 1 Pa = 0.000 145 lbf/in²):

Flexural Strength (kPa)	Estimated Compressive Strength (kPa)	Estimated Flexural Modulus (MPa)
210	840	175
285	1140	345
350	1400	690
500	2000	1380
650	2600	2050
950	3800	3500

Moduli values of 172, 345, 690, and 3445 MPa (25 000, 50 000, 100 000, and 500 000 lbf/in²) were considered.

A stress-dependent resilient-modulus behavior model was used to characterize the fine-grained subgrades. Thompson and Robnett (7) have developed a comprehensive data base and procedures for predicting the resilient behavior of Illinois soils. Stiff, medium, and soft subgrades were included in the study. Figure 1 shows the relations between resilient modulus and stress level for the various subgrades.

Thickness Levels

Soil-lime layer thicknesses of 15, 23, and 30 cm (6, 9, and 12 in) were included. For high-quality soil-lime mixtures, applications to low-volume roads, and typical highway loadings, layer thicknesses greater than 30 cm (12 in) are not common.

Loading

A wheel load of 40 kN (9000 lbf), half of the 80-kN [18 000-lbf (18-kip)] single-axle load commonly considered for design, was used in the study. A uniform tire pressure of 550 kPa (80 lbf/in²), distributed over a circular area, was applied in 138-kPa (20-lbf/in²) increments.

ILLI-PAVE Data

A summary of ILLI-PAVE soil-lime pavement responses is given in Table 1 for the range of soil-lime moduli, layer thicknesses, and subgrades considered. Figures 2-4 show relations among moduli, thickness, and flexural stress for stiff, medium, and soft subgrades, respectively. Relations between flexural strength and moduli, based on data given in the above text table, are also shown in the figures.

Development of Regression Equations

Because of computer capability requirements and cost considerations, ILLI-PAVE will not generally be available for use on a widespread basis. Even though Figures 2-4 can be used to approximate flexural stress for various conditions, it was considered essential to develop an algorithm for estimating soil-lime flexural stress for general use.

Several regression equations were developed to predict soil-lime flexural stress as a function of layer thickness, soil-lime modulus of elasticity, and subgrade resilient modulus. The best equation (based on the smallest standard error of estimate) is

$$\sigma_r = 23.22 - 4.66t + 42.36 \log E_B - 29.11 \log E_{Ri} \quad (1)$$

where

σ_r = flexural stress at the bottom of the soil-lime layer (lbf/in²),
t = thickness of soil-lime layer (in),
E_B = modulus of elasticity of the soil-lime layer (kip/in²),
E_{Ri} = resilient modulus of the subgrade (kip/in²) (Figure 1),
R = correlation coefficient = 0.95, and
$S_{\bar{x}}$ = standard error of estimate = 8.3 lbf/in².

A nomogram for solving Equation 1 is shown in Figure 5. Equation 1 should not be extrapolated beyond the range of parameters considered in this study.

Design Criteria

The shear and compressive strengths of cured soil-lime mixtures are not the limiting factors in their use as structural layers in the construction of low-volume roads. Soil-lime layers experience repeated flexural stresses, and therefore flexural strength and fatigue response are important considerations.

A typical fatigue-response relation for Illinois soils, obtained by averaging fatigue test results from previous studies (8), is shown in Figure 6 and can be expressed by the following equation:

$$S = 0.923 - 0.058 \log N \quad (2)$$

where S = stress ratio = repeated flexural stress/ flexural strength and N = number of load applications to failure.

Thompson (1) has emphasized that for many soil-lime mixtures the effect of continued strength development with increased curing tends to negate the effect of repeated loading. Many agencies use a "designated curing period" (fixed time and temperature conditions) for design of soil-lime mixtures (6). Quality criteria are based on mixture characteristics achieved by the soil-lime mixture cured in the specified manner. It is suggested that flexural strength and moduli should be determined in a like manner. Subsequent strength and modulus adjustments may be appropriate in some situations.

The design criteria for the soil-lime layer provide for adequate performance of that layer. Consideration of subgrade stresses should also be included in a comprehensive pavement design approach. The effects of soil-lime thickness, soil-lime-mixture modulus, and subgrade support are evident in Table 1.

USE OF DESIGN PROCEDURE

Required design inputs are soil-lime strength and modulus [the text table given earlier provides a general guide, and additional information is available elsewhere (6)], subgrade resilient modulus [information on testing procedures, estimating techniques, and other factors of influence is given elsewhere (7,9,10)], and traffic data (many procedures are available for calculating 80-kN equivalent single-axle loads). Flexural stress can be determined for a given thickness by using Equation 1, Figures 2-4, or the nomogram shown in Figure 5.

Calculate the stress ratio S, and predict the fatigue life based on Equation 2 or Figure 6 (8). Compare the predicted fatigue life with the estimated traffic to determine the adequacy of the design.

If significant soil-lime-mixture strength development (in excess of that assumed in design) is expected, a higher stress ratio may be acceptable. Factors such as freeze-thaw durability and curing conditions (time and temperature) should be considered in establishing a final thickness.

The design procedure is based on the concept of "flexural fatigue cracking". However, Suddath and Thompson (2) have demonstrated that the ultimate load-carrying capacity of

Figure 1. Resilient modulus versus repeated deviator stress for ILLI-PAVE analyses.

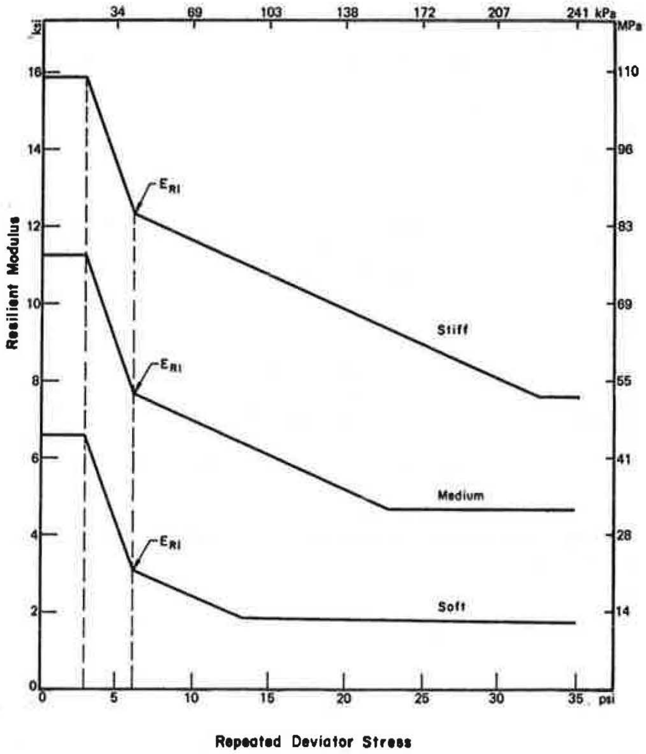


Figure 2. Relations among moduli, thickness, and flexural stress for stiff subgrade.

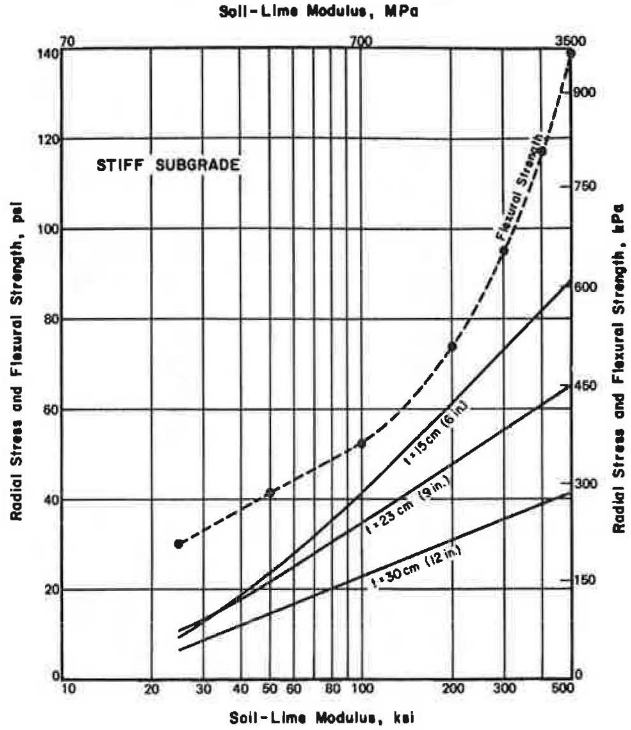


Table 1. Summary of ILLI-PAVE data for soil-lime layers.

Type of Subgrade	Thickness (cm)	Modulus (MPa)	Surface Deflection (mm)	Radial Strain[a]	Radial Stress (kPa)	Subgrade Vertical Stress (kPa)
Stiff	15	172	0.78	993	68	252
		345	0.69	814	165	206
		690	0.58	607	287	161
		3445	0.33	229	611	79
	23	172	0.58	678	74	167
		345	0.48	538	149	132
		690	0.41	388	238	101
		3445	0.23	135	452	46
	30	172	0.46	458	50	116
		345	0.38	358	99	92
		690	0.30	254	154	70
		3445	0.18	85	281	32
Medium	15	172	1.09	1389	129	220
		345	0.91	1051	238	176
		690	0.74	734	365	136
		3445	0.41	252	681	65
	23	172	0.79	906	116	145
		345	0.64	669	198	113
		690	0.51	455	287	85
		3445	0.28	146	486	38
	30	172	0.58	597	76	102
		345	0.48	436	127	79
		690	0.38	293	183	59
		3445	0.20	91	302	26
Soft	15	172	1.75	2210	255	166
		345	1.37	1468	365	132
		690	1.07	934	486	101
		3445	0.56	285	779	48
	23	172	1.17	1329	196	112
		345	0.91	879	276	86
		690	0.71	554	361	65
		3445	0.38	162	548	28
	30	172	0.86	837	120	81
		345	0.69	557	172	61
		690	0.53	352	227	45
		3445	0.28	98	329	20

Note: 1 mm = 0.039 in; 1 kPa = 0.145 lbf/in².

[a] μ strain.

Figure 3. Relations among moduli, thickness, and flexural stress for medium subgrade.

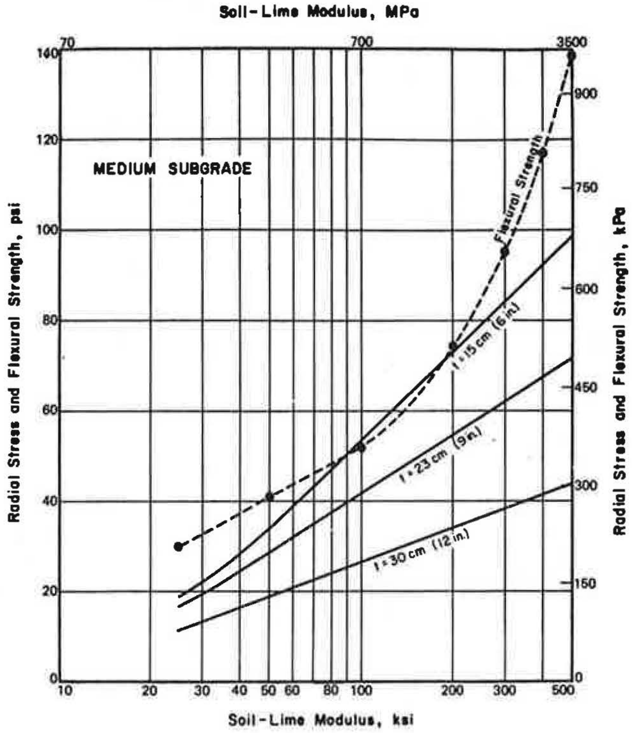


Figure 4. Relations among moduli, thickness, and flexural stress for soft subgrade.

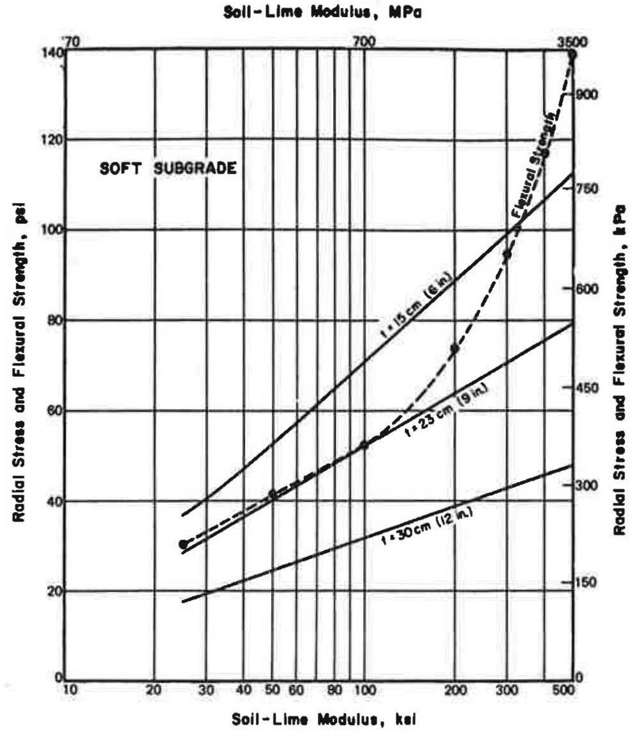


Figure 5. Nomogram for soil-lime flexural stress.

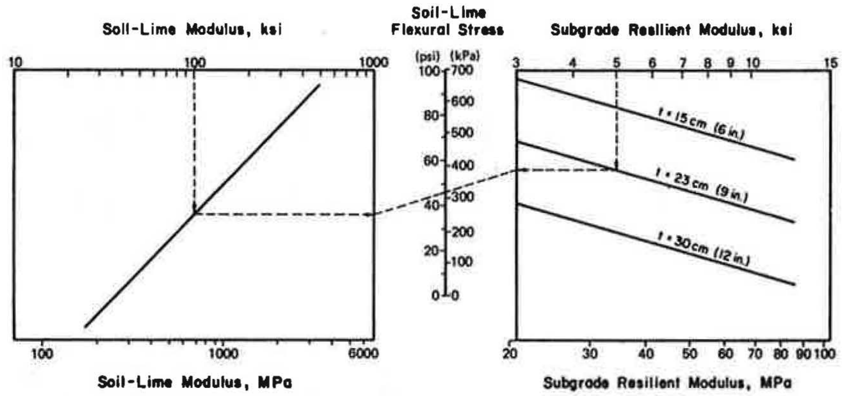


Figure 6. Fatigue response for cured soil-lime mixtures.

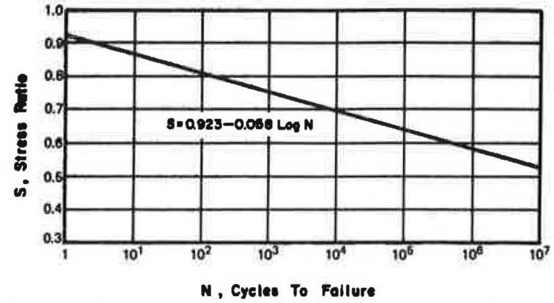
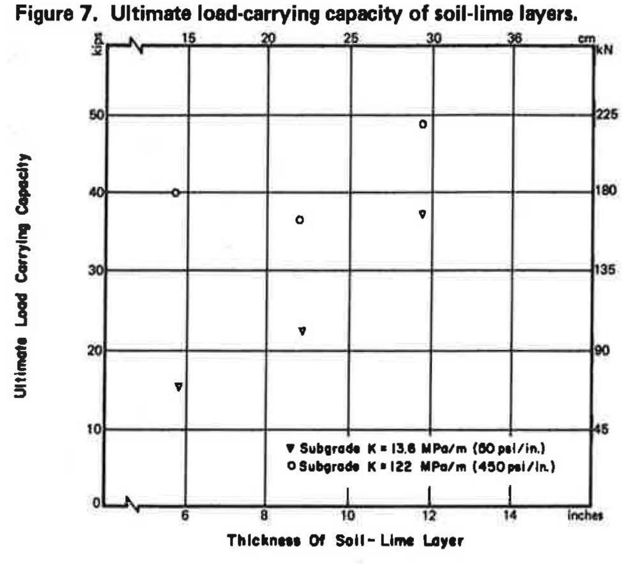


Figure 7. Ultimate load-carrying capacity of soil-lime layers.



soil-lime layers of normal thickness is considerably larger than the 40-kN (9000-lbf) wheel loading assumed in the development of the proposed design procedure (see Figure 7). Thus, the pavement does not fail in a catastrophic manner, even though a so-called flexural fatigue crack develops.

SUMMARY

A mechanistic thickness-design procedure for cured soil-lime structural paving layers is presented. Design input data are soil-lime strength and modulus, subgrade resilient modulus, and estimated traffic. The procedure assumes that the soil-lime mixture is capable of developing significant strength increase (relative to the strength of the natural soil) and that quality field construction and control are achieved.

It is emphasized that careful consideration must be directed to the proper selection of design input data, particularly soil-lime properties and subgrade resilient modulus. Additional factors, particularly freeze-thaw durability, may also need to be considered in some applications.

The procedure considers the thickness of the soil-lime layer only. Additional consideration of subgrade stress is required in a comprehensive approach to pavement design.

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Use of Cement-Kiln Dust and Fly Ash in Pozzolanic Concrete Base Courses

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The results of a study to determine the usefulness, as stabilized base, of pozzolanic concrete that contains cement-kiln dust (CKD) and fly ash are reported. Test strips of six different mixes of pozzolanic concrete that contained CKD and fly ash as the cementitious ingredients and crushed limestone as the aggregate were constructed on a concrete plant drive at Silica, Ohio. Deflection measurements, monitoring of axle-load accumulations, and periodic compression tests of field samples were performed. Laboratory test cylinders were also prepared and tested. After 25 820 equivalent 80-kN (18 000-lbf) single-axle loads over six months, no cracking or surface damage was visible except in a localized area. Deflection was found to decrease as curing time for the test strips increased. A set of regression equations was developed for predicting laboratory compressive strengths as a function of CKD content, curing temperature, and curing time in weeks. Application of these equations to field conditions resulted in reasonable predicted, as compared with observed, field strengths in most cases. Pozzolanic concrete containing CKD and fly ash as cementitious ingredients was found to have the property of autogenous healing. Anomalies were found in mixes that contained admixtures of portland cement. More study of the behavior of such mixes is needed. It is concluded that pozzolanic concrete that contains CKD and fly ash is potentially useful as stabilized base and merits further development.

Stabilized pavement bases are considered by many engineers to offer advantages with respect to load-carrying capacity and resistance to climate-induced deterioration. Various materials have been used as binders in stabilized bases, including asphalt, portland cement, and lime-fly ash mixtures.

Lime-fly ash mixtures, which are well known in Illinois, Ohio, Pennsylvania, and other states, use a waste product as one of two dry ingredients that react in the presence of water to form a "pozzolanic cement". This waste product is fly ash collected from the stacks of coal-burning utilities. In addition to the structural advantages of a stabilized pavement base, use of a waste product as part of the cementitious binder of such a base can produce other beneficial results, including

1. Lower construction cost because of the lower prices for ingredients developed from waste products,
2. Energy savings as a result of the elimination of the necessity for environmentally sound waste-disposal procedures, and
3. Reduced energy consumption for the construction of pavements, since relatively little energy is required in the preparation of paving ingredients made from waste products.

Potential energy-related benefits can be expected to become increasingly important if the cost of energy continues to rise or if petroleum shortages persist. Thus, from the standpoint of energy conservation, it would be useful if a pozzolanic concrete could be developed that would use some waste product in place of lime, a material that requires a rather energy-intensive manufacturing process. Such a pozzolanic material would thus be formed of two waste products. It is, of course, essential that the properties of any new pozzolanic cement be comparable to those of well-established binding materials for stabilized bases. No reduction in the standards for pavement performance or durability can be contemplated in the name of energy conservation unless the energy problem becomes substantially more acute than it is at present.

Experiments have shown that the dust collected from the stack of cement kilns can react with fly ash in the presence of water to form a cementitious material that has physical properties comparable to those of lime-fly ash. Cement-kiln dust (CKD) is available in large quantities because all cement plants must collect their emissions to satisfy air pollution regulations. Although CKD is not without some potential usefulness for purposes other than paving, the quantities of CKD that are generated are sufficient to present the cement industry with serious disposal or storage problems.

If, therefore, CKD can be used for pavement construction, it should be possible to achieve some environmental enhancement through reduction of waste stockpiles and some economic advantage to the cement industry through the transformation of significant quantities of CKD from liabilities to assets. Such achievements would be in addition to the energy conservation benefits that would accrue to the general public. It is the purpose of this paper to describe field and laboratory tests of pozzolanic concrete in which CKD and fly ash were used as cementitious ingredients.

COMPOSITION OF CEMENT-KILN DUST

Since it is a waste product, cement-kiln dust is by nature somewhat variable in its chemical composition and in the properties associated with that composition. Data in the table below, which were developed from analyses of CKD from nine separate sources, describe in general terms the chemical analysis of CKD:

Ingredient or Property	Percentage Low	Mean	High
SiO_2	6.0	16.5	28.5
Al_2O_3	3.2	4.4	9.6
Fe_2O_3	0.8	2.7	5.9
CaO			
From 9 sources	16.0	47.6	65.0
From 13 sources	8.3	12.6	20.2
MgO	0.8	2.3	4.8
SO_3	0.7	7.1	26.3
Na_2O	0.08	0.8	3.2
K_2O	1.08	5.5	26.2
Loss on ignition	2.50	16.0	32.0

Some CKD has cementitious properties in and of itself. But the data given in Table 1 illustrate the greater compressive strength attainable from a pozzolanic combination of CKD and fly ash. The cubes described in Table 1 were made from batches that contained 1.36 kg (3 lb) of sand and various quantities of other ingredients, as indicated in the table. An attempt was made to achieve a uniform consistency by determining the penetration of a Vicat needle. Water was added in sufficient quantity to yield a mixture capable of easy compaction in the cube molds. The cubes were all 5.08x5.08x5.08 cm (2x2x2 in) in size. In each case given in Table 1, it can be seen that the strength of the cubes that contained both CKD and fly ash was about 1.9 times the strength of the cubes that contained CKD alone.

DESCRIPTION OF THE STUDY

Several experimental sections of pozzolanic concrete containing fly ash and CKD have been placed in the vicinty of Toledo, Ohio, and elsewhere. All of these pavements are being observed. Detailed tests and field observations have been performed at a concrete batching plant located in Silica, Ohio, that belongs to Nicholson Industries of Toledo. The subject of this paper is the program that was carried out at that site between November 1977 and December 1978.

The site at Silica provided an opportunity to observe the performance of test sections under heavy but controlled traffic. The test sections were placed in the driveway leading away from the concrete mixer so that the numbers of 80-kN (18 000-lbf) equivalent single-axle loads could be closely estimated by analyzing the concrete-truck "trip tickets" and a knowledge of the axle configurations of the various trucks in service.

Six test strips were placed at Silica, each about 30.5 m (100 ft) long. Compositions of the six different mixes used are given below:

Test Strip	Percentage by Weight of Dry Ingredients: Fly Ash	CKD	Portland Cement	Aggregate
1	6	6	0	88
2	8	8	0	84
3	10	10	0	80
4	12	12	0	76
5	8	8	0.5	83.5
6	8	8	1	83

The fly ash used came from the Detroit Edison Company power plant in Trenton, Michigan; the CKD came from the Medusa Cement Company at Silica, Ohio; and the aggregate was Ohio specification 301 crushed limestone.

Moisture contents were designed to be optimum for the mixes in question. Field moisture contents and densities, obtained by using a nuclear meter, are given below. Mix identification numbers refer, respectively, to percentage fly ash, percentage CKD, and percentage portland cement by dry weight (1 kg/m³ = 0.062 lb/ft³):

Mix	Dry Unit Weight (kg/m^3) Layer 1	Layer 2	Moisture (%) Layer 1	Layer 2
6-6-0	1913	1904	12.2	11.0
8-8-0	1992	1982	14.1	14.0
10-10-0	1890	1949	11.0	9.7
12-12-0	1898	1914	11.0	9.5
8-8-0.5	1929	2006	9.9	9.8
8-8-1	1925	1958	9.8	9.6

The test strips were all placed at a total thickness of 25.4 cm (10 in) in two layers of 12.7 cm (5 in) each. The test-strip surfaces were given a double "tar-and-chip" seal coat after placement. All base material was placed on November 5, 1977, quite late in the season for northern Ohio.

Soil borings and mechanical analyses of the subgrade were made the following spring, on May 6, 1978. The subgrade material was found to consist largely of brown sandy silt, classified for the most part as A-4a in the Ohio Department of Transportation system and possessing a group index of about 8.

Planned research work on the test strips included the following:

1. Field observations—(a) deflection measurements by Benkelman beam, (b) monitoring of cracks, rutting, or other surface anomalies, (c) monitoring of axle-load accumulations, and (d) recording of air temperatures at the nearest meteorological station (Toledo Express Airport);
2. Laboratory work—(a) compression tests and density observations of samples collected from the field and (b) preparation of laboratory specimens for curing at various

Table 1. Results of cube compression tests of CKD-fly ash mixtures.

Mix	Location of CKD Source[a]	Batch Quantity (kg) CKD	Fly Ash	Water	Vicat Penetration (mm)	28-Day Compressive Strength (kPa)
1	Tennessee	1.36	0	0.48	6.5	7 410
2	Tennessee	0.91	0.45	0.45	7	14 020
3	Florida	1.36	0	0.46	10	4 895
4	Florida	0.91	0.45	0.43	10	9 295

Note: 1 kg = 2.2 lb; 1 mm = 0.039 in; 1 kPa = 0.145 lbf/in².
[a]General Portland, Inc.

Table 2. Results of Benkelman beam observations.

Mix	5/16/78 Measurement (mm) Avg Deflection	Standard Error	6/17/78 Measurement (mm) Avg Deflection	Standard Error	9/16/78 Measurement (mm) Avg Deflection	Standard Error
6-6-0	0.44	0.20	0.32	0.08	0.23	0.03
8-8-0	0.52	0.24	0.55	0.08	0.70	0.11
10-10-0	0.79	0.22	0.79	0.26	0.53	0.13
12-12-0	0.29	0.13	0.42	0.20	0.27	0.20
8-8-0.5	0.61	0.04	0.52	0.06	0.27	0.10
8-8-1	0.86	0.05	0.16	0.10	0.16	0.08

Note: 1 mm = 0.039 in.

Table 3. Results of laboratory tests of compressive strength.

Time Cured (weeks)	Curing Temperature (°C)	Compressive Strength (kPa) Mix 6-6-0	Mix 8-8-0	Mix 10-10-0	Mix 8-8-0.5	Mix 8-8-1
1	37.8	1938	3848	4007	6669	7676
	10.0		0		986	1600
2	37.8	3690	5745	6386	6372	8469
	10.0		910		1421	2138
4	37.8	4538	6441	8048	4786	9159
	10.0		1366		2793	4055
12	37.8	5862	7910	8276	8441	8248
	10.0		6821		6476	7628

Note: t°C = (t°F − 32)/1.8; 1 kPa = 0.145 lbf/in².

temperatures, to study compatibility of field and laboratory results (curing and testing of laboratory specimens in accordance with ASTM C593-76a except for variable temperature).

In April 1978, a careful and detailed topographic survey was carried out to serve as a basis for studying ruts or other surface problems.

STUDY RESULTS

A total of 25 820 equivalent 80-kN (18 000-lbf) single-axle loads were recorded over the test areas during the first six months of their existence. Many visual observations during the first year revealed no visible cracking, rutting, or other degradation of the surface except for a small area near the loading station for concrete trucks, where water is frequently sprayed from a hose and the pavement is wet most of the time. The test strips were placed very late in the year, and the winter of 1977-1978 proved to be a particularly severe one in northwest Ohio. On only 23 days between November 5, 1977, and May 13, 1978 (when the first samples were collected for strength tests), did the temperature average at least 10°C (50°F). During the same period, there were only 53 days on which the temperature averaged at least 4.4°C (40°F). The pozzolanic reaction proceeds slowly at 10°C and probably stops at temperatures below 4.4°C. Clearly, therefore, the test strips at the Silica site were subjected to a severe test during their first winter.

Results of the Benkelman beam deflection tests are given in Table 2. Clearly defined trends and uniform conditions were not encountered in the course of these deflection measurements. Probable causes of the comparatively wide range in results are high bedrock elevations and lack of uniformity of the subgrade. Bedrock lies within several feet of the surface throughout the test area, and local outcrops are visible at some points. With the exception of the 8-8-0 section, it can be observed that deflection generally decreases with time. Test loads for deflection measurements were obtained by filling a concrete truck with crushed stone and weighing the test axle. The test axle in every case was a single axle with dual wheels, at approximately 80 kN.

Compression test results (for an average of three samples) from samples collected in the field are summarized below (1 kPa = 0.145 lbf/in²):

Mix	Compressive Strength (kPa) 5/10/78	5/17/78	10/16/78	10/17/78
6-6-0			8 122	
8-8-0		4220	10 618	
10-10-0		4213		9915
8-8-0.5		1731	8 487	
8-8-1	5592			2448

The 12-12-0 mix was not tested because it was concluded that such a mix would not be feasible to construct under ordinary job conditions. Samples were obtained in the field by sawing blocks from the pavement and extracting them with picks or bars. These pieces were then sawed in the laboratory to blocks of more convenient size and regular dimensions. Coherent field samples of mixes not containing some portland cement could not be obtained as early as May 10, 1978. The typical specimen size was 10 cm (4 in) in height by cross section about 9 cm (3.5 in) square. No corrections were made for the ℓ/d ratio, which averaged 1.1 ± 0.2.

Laboratory compression test results are presented in Table 3. The effects of curing time and curing temperature can be observed from these results.

DISCUSSION OF RESULTS

Curing Time and Compressive Strength

It was found that the compressive strengths of cylinders made in the laboratory could be predicted quite accurately in terms of curing time and temperature, expressed as degree-weeks. A degree-week is defined for this purpose as a curing temperature maintained 1° above some reference temperature for a period of seven days (168 h). As previously mentioned, the chemical reaction between CKD and fly ash (the pozzolanic reaction) proceeds slowly at temperatures below 10°C (50°F) and probably not at all at temperatures below 4.4°C (40°F). It was found by trial that the selection of a reference temperature of 7.2°C (45°F) yielded the best fit to the data.

Statistical analysis showed that the relation between the compressive strength of laboratory specimens and degree-weeks could be depicted by an equation of the following form:

$$Y = b \log_{10} X + a \tag{1}$$

where

Y = compressive strength (kPa),
X = accumulated degree-weeks, and
a and b = numerical coefficients.

Regression coefficients for an expression having the form of Equation 1 are given below for each of the mixes investigated in the laboratory (1 kPa = 0.145 lbf/in²):

Mix	a	b	Correlation Coefficient	Standard Error of Estimate (kPa)
6-6-0	-2930	3500	0.981	±526
8-8-0	-1794	4054	0.936	±998
10-10-0	-987	3868	0.904	±731
8-8-0.5	-657	3937	0.944	±899
8-8-1	409	3909	0.903	±1221

Correlation coefficients are also given above, and it can be seen that these coefficients are quite high. Figure 1 shows an example plot of compressive strength as a function of degree-weeks. Figure 2 depicts all five regression lines and their relations to one another and demonstrates that the rate of strength gain is about the same for all mixes. It is also apparent from Figure 2 that the regression lines fall in the order to be expected if strength is an increasing function of CKD content and if an admixture of portland cement contributes further to compressive strength.

Prediction of Field Compressive Strength from Laboratory Data

The regression equations given in the preceding text table were applied to observed air temperatures and strength data from test-strip samples in an attempt to test the predictive value of such equations under field conditions. Figure 3 shows the accumulated degree-weeks at the Silica site, based on a reference temperature of 7.2°C (45°F) and mean daily temperatures as recorded at Toledo Express Airport. Since the test strips were covered only by a double seal coat, the upper portions of the test strips were probably close to ambient air temperature. Lower portions of the pavement may have been warmer, but samples for testing were taken only from the upper layers. But the fact that most observed strengths were higher than predicted may be the result of higher temperatures within the pavement layers, in comparison with the ambient air temperature.

The results of applying the regression equations to the data from the field are given below (1 kPa = 0.145 lbf/in²):

Mix	Cumulative Degree-Weeks	Compressive Strength (kPa) Predicted	Compressive Strength (kPa) Observed	Observed/ Predicted (%)
6-6-0	292	5 702	8 122	142
8-8-0	24.3	3 827	4 220	110
	292	8 205	10 618	129
10-10-0	24.3	4 371	4 213	96
	292	8 549	9 915	116
8-8-0.5	24.3	4 488	1 731	39
	292	9 039	8 487	94
8-8-1	17.0	5 219	5 592	107
	292	10 046	2 448	24

It can be seen that consistent results were obtained, with the exceptions of the early data for the 8-8-0.5 mix and

Figure 1. Example regression line for compressive strength versus degree-weeks for 8-8-1 mix.

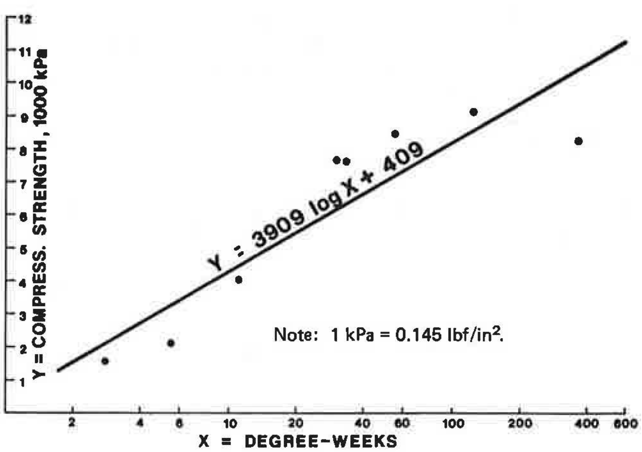

Figure 2. Regression lines for the five mixes studied in the laboratory.

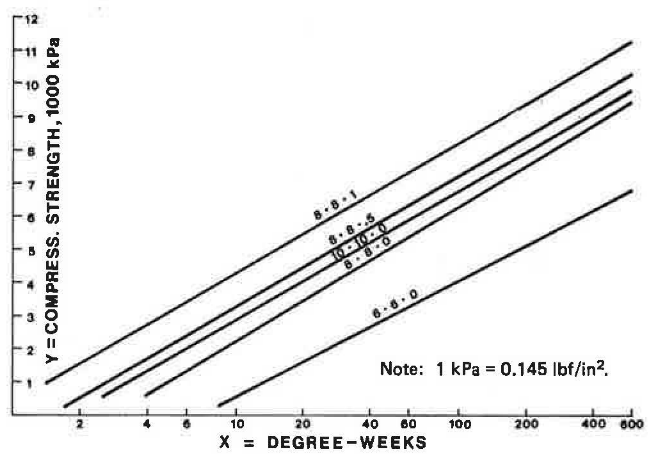

Figure 3. Accumulated degree-weeks at Silica site based on accumulated degree-days recorded at Toledo (reference temperature of 7.2°C).

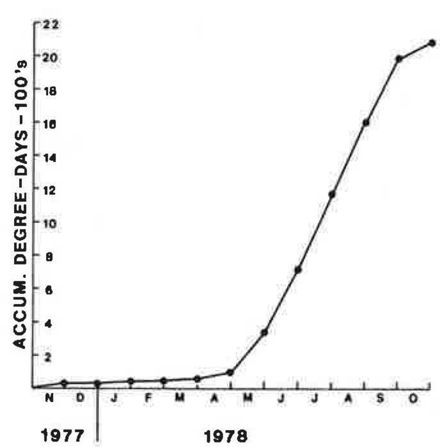

Table 4. Data on autogenous healing of laboratory samples.

	Original Test Condition			Retest Condition		
Mix	Curing Tempera-ture (°C)	Cure Duration (weeks)	Compressive Strength (kPa)	Healing Time (weeks)	Compressive Strength (kPa)	Retest Condition/ Original Condition
6-6-0	37.8	12	5860	7.3	6 564	1.12
		4	4537	10.9	5 764	1.27
		1	1937	6.3	3 089	1.60
8-8-0	37.8	12	7908	7.3	7 936	1.00
	10.0	12	6819	3.9	10 390	1.52
	37.8	4	6440	10.9	7 701	1.20
		1	3847	9.9	5 137	1.34
10-10-0	37.8	12	8274	7.3	8 363	1.01
		4	8046	10.9	9 694	1.20
		1	4006	5.3	5 019	1.25
8-8-0.5	37.8	12	8439	5.7	10 225	1.21
	10.0	12	6474	3.9	8 687	1.34
	37.8	1	6667	12.6	8 467	1.27
	10.0	1	1034	12.6	6 943[a]	6.71
8-8-1	37.8	12	8246	5.7	10 501	1.27
	10.0	12	7626	3.9	11 831	1.55
	37.8	1	7674	12.6	10 170	1.33
	10.0	1	1600	12.6	8 942	5.59

Note: $t°C = (t°F - 32)/1.8$; 1 kPa = 0.145 lbf/in².
[a] Average of two cylinders.

later for the 8-9-1 mix. It can be seem from the third text table on page 37 that all mixes placed in the field were of about the same dry unit weight.

Autogenous Healing

Cementitious compounds that are formed by chemical reactions proceeding at relatively slow rates may possess the property of autogenous healing. That is, cracks that may be formed in such compounds and that are not too wide may "heal" as a result of continued chemical activity. Laboratory cylinders made for the project were stored in a room at 21.1°C (70°F) and 50 percent humidity for varying periods after having been broken for the first time. Nothing was done to these cracked cylinders during the period of storage after their first compression test and, when they were retested, they were not given the 4-h soak required by ASTM C593. The original caps were left on the specimens for use at retesting.

Table 4 presents the results of autogenous-healing tests. The typical retest strength can be seen in that table to be about 1.3 times the original compressive strength. This ratio (retest strength/original strength) appears not to vary with the duration of healing prior to retest. The two cases in which the ratio was substantially higher than 1.3 are both associated with initial curing temperatures of 10°C (50°F) in which initial compressive strength was exceptionally low. With the one exception noted in the table, the data presented in Table 4 are all derived from the means of three cylinders for each test.

ADDITIONAL RESEARCH REQUIRED

Some anomalies were observed in the results of tests on field samples that contained portland cement. The results of the compression tests were much lower than had been anticipated. It will be noted that the strength of the 8-8-1 mix actually decreased over a five-month period. The same phenomenon was observed in the laboratory in the case of 8-8-1 specimens cured at 47.8°C (100°F) for longer than four weeks. Although the addition of portland cement clearly contributes to the early strength of the material, more study is needed to identify the effect of that cement on the pozzolanic reaction between the CKD and the fly ash.

Detailed examination revealed many small cracks in a layer of material at the surface of each block sawed and extracted from the test strips. In addition, the aggregate was found to have become degraded in this layer, the thickness of which varied from 2.5 to 5.1 cm (1-2 in). The same aggregate was used for the laboratory studies, in which some degradation under the compaction hammer was also observed. It is postulated that the tar-and-chip double seal coat was an insufficient wearing surface under the heavy traffic on the test strip and that traction forces caused the degradation of the limestone aggregate. But more study is needed to gain an adequate understanding of the behavior of CKD pozzolanic concrete under tractive forces.

More data are required on the deflection properties of CKD pozzolanic concrete, and its stress-strain characteristics must be established. Further verification of the regression models for strenth prediction, as well as investigation of the behavior of fly ash and CKD from a variety of sources, would be desirable.

CONCLUSIONS

The results of the field and laboratory work described in this paper are considered to demonstrate the potential usefulness, as a stabilized base, of pozzolanic concrete that contains cement-kiln dust and fly ash as the cementitious ingredients. Other conclusions are as follows:

1. Compressive strengths of laboratory cylinders can be predicted satisfactorily by means of regression equations relating to strength of CKD content, curing temperature, and curing time.
2. The results of the work indicate that the regression equations mentioned above can also predict safe estimates of field compressive strength by using ambient air temperature. But more study of the effect of environment and mix design on field strengths is needed. Data on the thermal properties of CKD mixes are needed to estimate pavement temperatures with sufficient reliability to use such temperatures in field strength prediction.
3. Pozzolanic concrete that contains CKD and fly ash as the cementitious ingredients has been shown to possess the property of autogenous healing.
4. More study is required to establish the role of portland cement as an admixture in CKD pozzolanic concrete.
5. Further research and development, and especially further study of field performance in experimental pavements, are justified by the results of the work reported here.
6. The results apply to the materials used in the study. Use of any particular combination of CKD and fly ash requires that chemical analyses and compression tests be performed to establish the properties of the specific materials in order to develop suitable mix designs.

ACKNOWLEDGMENT

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Dependence of Compacted-Clay Compressibility on Compaction Variables

ALBERT DiBERNARDO AND C. W. LOVELL

Conventional oedometer tests were performed on a kneading-compacted, highly plastic residual clay to determine the effects of water content, dry density, and compaction pressure on the as-compacted and soaked compressibility behavior of the clay. Environmental changes caused by increased saturation were simulated by using a back-pressuring technique. Of interest was the one-dimensional percentage change in volume on wetting under load. Statistical analyses were performed to establish the best predictive model for one-dimensional percentage change in volume on saturation in terms of the important compaction variables. The statistical model shows that initial water content and nominal compaction pressure are the principal variables that affect this response. Such equations allow prediction and control of the compressibility of high, cohesive embankments.

Compacted soils are used primarily for the construction of earth embankments. During design, the engineer must establish suitable placement variables, such as water content, dry density, and compaction effort, to ensure adequate short- and long-term performance of the completed structure. Typically, the shear strength of the compacted mass is of major concern, and recent studies (1-3) have provided the capability of predicting its laboratory and field response value in terms of the important compaction variables. But the demand for high embankments is increasing, and it is becoming increasingly more important to specify compaction procedures so that embankment compressibility can be adequately predicted and controlled in both the as-compacted and long-term conditions.

A compacted soil is a three-phase system that consists of soil grains or aggregations, water, and air. During the compaction process, densification is achieved by a reduction in air voids at constant water content. The influence of this procedure on compressibility behavior depends considerably on (a) the compaction water content, (b) the amount of compaction, and (c) the compaction mode. All of these factors affect the fabric of the compacted clay. In service, this fabric may be altered over time because of environmental changes, which, in turn, may effect a change in compressibility behavior.

The research reported here was conducted to examine the compressibility behavior of a laboratory-compacted soil in the as-compacted and saturated condition. The type of soil used was a highly plastic residual clay, and the mode of compaction was of the kneading type.

To determine the as-compacted compressibility characteristics, the compacted samples were trimmed to the appropriate size and incrementally loaded in an oedometer. To simulate the changes in mass that may occur in service, compacted specimens were saturated in the oedometer under an equivalent embankment load. A back-pressuring technique was used to achieve full saturation of the oedometer specimens. Of major interest was the percentage of volume change that occurred during saturation under loading.

The ultimate purpose of this research was to establish models that can be used to accurately predict laboratory compressibility characteristics in terms of the important compaction variables. The models proposed will provide the engineer with a tool for controlling as-compacted and long-term compressibility.

A similar study on the compressibility characteristics of field-compacted samples of the same soil is in progress. The combined results of both studies will allow the engineer to establish suitable compaction specifications for controlling such behavior.

BACKGROUND INFORMATION

Yoshimi (4) studied the physical and mechanical mechanisms that control the consolidation of unsaturated soil. He divided the total compression process into three stages—initial compression, consolidation, and creep—and assumed that all three stages occur singly and in the stated order. These stages are defined as follows:

1. Initial compression—the immediate response caused by the compression of soil fabric and gas as soon as the incremental load is applied,
2. Consolidation—that part of compression that involves the outflow of pore fluids, and
3. Creep—that part of compression that involves the redistribution of shearing stresses in the absorbed water and the local rearrangement of the soil particles.

In 1974, Barden (5) explained the volume-change behavior of compacted clays as follows. In the dry-of-optimum condition (continuous air voids),

> Assuming that the initial value of pore air pressure (u_a) is atmospheric the initial suction will cause the value of pore water pressure (u_w) to be negative .... On applying a load to the soil there will be an initial or immediate compression, and because of the relative stiffness of the soil skeleton and the highly compressible pore-fluid, the values of Δu_a and Δu_w will be small. Thus there will be an increase in u_a and u_w, but in general u_w will remain below atmospheric pressure and hence only air can drain from the soil. Even in cases where u_w does rise above atmospheric pressure, the value of air permeability (k_a) is so much greater than the water permeability (k_w) that the flow must be completely dominated by the air ....

In the wet-of-optimum condition (occluded air bubbles),

> The material develops pore water pressure, increasing with water content. Since this is also associated with a low value of k_w, consolidation in the classical sense is a real engineering problem involved in the stability analysis of embankments, etc. It has been shown that despite the expansion of air bubbles, the variation of permeability during a consolidation stage is no more marked than in many saturated clays. It is also shown that the compressibility of the pore fluid should not be a particularly important factor, provided the

coefficient of volume compressibility (m_v) is defined with respect to the overall settlement. Thus on theoretical grounds it appears that (classical) Terzaghi theory should prove adequate.

In the transition from dry to wet, Barden indicates that, right up to occlusion, $k_a >> k_w$ and no water flows. He also suggests that occlusion occurs at roughly optimum water content, although there may be regions of continuous air voids with low values of k_a, and that the occlusion process and the transition from a dry to a wet process are sudden rather than gradual.

Hodek (6) explained the characteristics and engineering behavior of a compacted soil in terms of a deformable aggregate model. According to Garcia-Bengochea (7), measurements of pore-size distribution for compacted clays have also provided strong evidence for a deformable aggregate model.

EXPERIMENTAL APPARATUS AND PROCEDURE

The soil used in this study was obtained from the IN-37 relocation project in Perry County, Indiana, and is referred to as St. Croix clay. The soil is a highly plastic residual clay from sandstone-shale parent material. Its properties are described below:

Category	Classification or Value
Soil classification	
Unified	CH
AASHTO	A-7-6
Index properties (%)	
Liquid limit, w_ℓ	53
Plastic limit, w_p	21
Plasticity index, I_p	32
Shrinkage limit, w_s	12
Specific gravity, G_s	2.80
Natural moisture, w_n (%)	20
Clay-size fraction < 2 μm (%)	44

An X-ray diffraction analysis was performed on the clay-size fraction; kaolinite was found to be the predominant clay mineral, but montmorillonite was also present in trace amounts.

The kneading method of compaction was used in this study. Typically, however, density and water content are the prescribed elements in an end-result approach and are based on results obtained by the laboratory impact method. It was therefore considered appropriate to use the relations among impact, moisture, and density as a base reference for the kneading method.

The moisture-density curves for the three basic levels of impact energy used are shown in Figure 1. The compaction assembly and procedure were as specified in AASHTO T99-70, but fresh samples were used for each point on the curve. Kneading-compacted samples were fitted to these curves by adjusting the foot pressures.

A Karol-Warner fixed-ring consolidation cell was used in this study. The oedometer ring is 63.5 mm (2.50 in) in diameter, 2.54 mm (1 in) in height, and 19.1 mm (0.75 in) in wall thickness. The loading system used to compress the sample is a lever arm-weight type.

A tube sample was taken from the compaction mold (8) and hydraulically extruded directly into the oedometer ring. The upper and lower faces of the specimen were trimmed with a steel straightedge by using the top and bottom of the ring as guides.

Following a seating-load-adjustment period (typically 10 min), the applied pressure was increased, by using a load-increment ratio (LIR) of 0.5, to 15.2, 22.6, 34.0, 49.5, 76.5 kPa (2.2, 3.3, 5, 7.2, and 11 lbf/in²), etc. The duration of each load was 10 min, during which dial readings were typically recorded at 0, 0.1, 0.5, 1, 2, 4, 8, and 10 min.

The 10-min load-duration criterion was based principally on the findings of Yoshimi (4). To determine which LIR to use for the as-compacted test, it was decided that the following requirements should be met:

1. A large portion of the compression must take place within 10 min,
2. As few loads as possible would be used to limit moisture loss by evaporation, and
3. The compaction prestress value could be accurately defined by using the Casagrande construction.

Based on the results of Leonards and Girault (9) for determining the preconsolidation value for saturated soils, if an LIR of 1 were used, requirements 1 and 2 above would be satisfied but requirement 3 might not. If an LIR of 0.1 or 0.2 were used, requirement 3 would be satisfied but not requirements 1 and 2 (4). Ideally, an LIR equal to actual field loading conditions should be used; it is dependent, however, on a given application and may be difficult to determine. Therefore, an LIR of 0.5 was chosen because it most suitably matched the requirements.

It is likely that the volume changes caused by saturation are important. To measure these, the compacted samples were compressed (LIR = 0.5, load duration = 10 min) until a vertical pressure of either 160, 320, or 480 kPa (23, 46, or 70 lbf/in²) was applied. The samples were then saturated by a de-airing and back-pressuring procedure. The consolidating loads—either 160, 320, or 480 kPa—represent the pressure exerted by an equivalent embankment height of 7.8, 15.6, or 23.5 m (25, 50, or 75 ft), respectively.

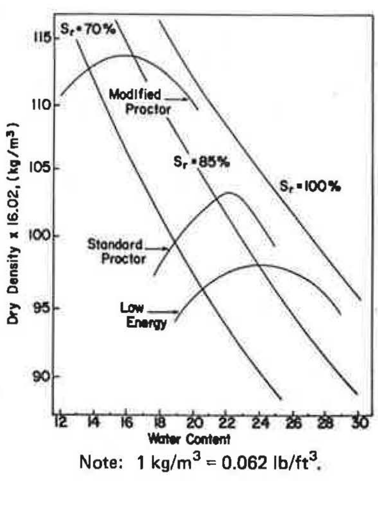
RESULTS AND DISCUSSION

As-Compacted Compressibility

Compressibility tests were labeled as follows: to correspond to the equivalent low energy (L), standard Proctor (S), and modified Proctor (M) impact levels; dry-of-optimum (D), optimum (O), or wet-of-optimum (W) conditions; and sample numbers 1, 2, etc. For example, MD1 characterizes the first sample compacted to a dry-of-optimum condition by using equivalent modified Proctor kneading-compaction pressure.

Figure 2 shows the relative compression (compression at time = t divided by total compression at time = 10 min) versus time. For the dry-side and optimum samples (SD8, SO16), the magnitudes of relative compression are virtually the same at each successive time plotted. It is believed that this would not occur if the compression of both samples were not dominated by the outflow of pore air (Δu_w < 0)—specifically, if the air voids were not interconnected. Figure 2 also shows the magnitude of relative compression with time for an initially wet-of-optimum sample (LW2). In comparison, the wet-side sample exhibits the least amount of relative compression within the 10-min period.

Figure 1. Moisture-density-energy relations for three levels of impact energy.

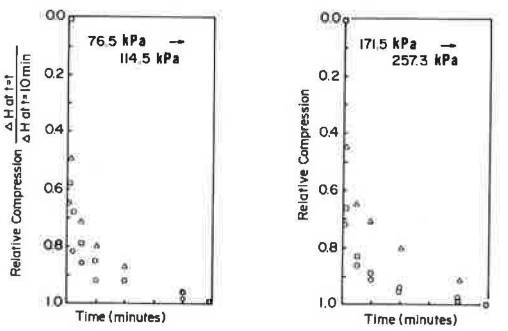


Note: 1 kg/m³ = 0.062 lb/ft³.

The practical significance of these data is that the settlement attributable to as-compacted compressibility is likely to be achieved during the construction period.

Figure 3 shows the effects of increasing water content and degree of saturation on compressibility behavior for samples compacted to equivalent low-energy conditions. There is a marked difference in the compressibility behavior for wet- and dry-side samples depending on the range of consolidation pressure considered; that is, in the low-pressure range [20-200 kPa (3-30 lbf/in²)], the wet-side sample is more compressible than the dry-side sample, whereas in the high range (>200 kPa) the opposite is true. Similar behavior is observed for wet- and dry-side standard Proctor samples and for wet- and dry-side modified Proctor samples.

Figure 2. Comparison of relative compressions for at-optimum, dry-of-optimum, and wet-of-optimum samples.



Note: 1 kPa = 0.145 lbf/in².

Figure 3. Effect of moisture content on compressibility: low-energy condition.

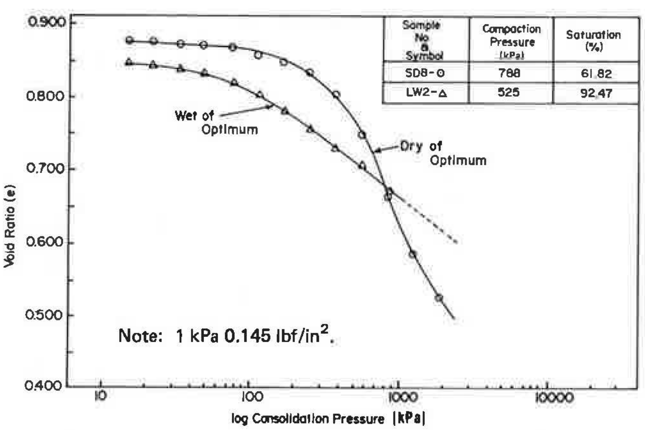
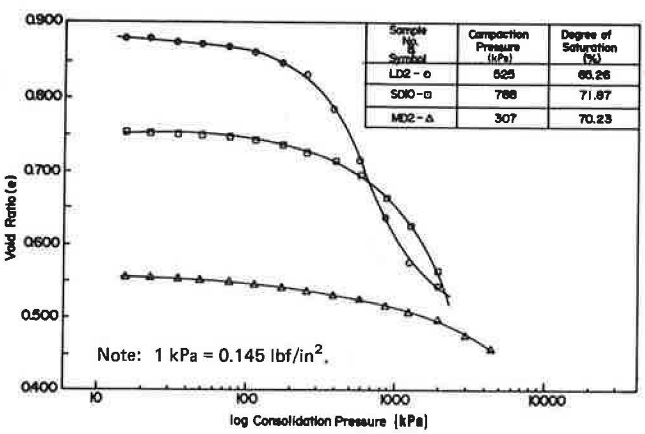


Figure 4. Effect of compactive effort on compressibility: dry of optimum.



Recently, Hodek and Lovell (10) examined this behavior in terms of pore-size distribution and magnitude and the deformable aggregate theory. Based on supporting evidence (6,7,11), their explanation can be stated as follows:

1. Dry-side compressibility—The pores are typically large and numerous; the clay aggregates are shrunken, hard, and brittle; and compressibility is governed by the collapse of large pores under straining; and
2. Wet-side compressibility—The pores are small and numerous, the clay aggregates are swollen and plastic, and compressibility is governed by the fusing of aggregates under load.

In view of this explanation, a dry-side sample would compress less in the low-pressure range because of the large intergranular forces that result from the many well-developed meniscii. However, on loading, these forces are overcome, and the brittle aggregates displace into adjacent pores. Consequently, a large amount of compression occurs because of the large amount of available interaggregate pore space. On the other hand, a wet-side sample compresses more in the low-pressure range because of the smaller number of developed meniscii (high degree of saturation) and less in the higher range as a result of the relatively small number of large interaggregate pores.

Figure 4 shows the effect of increasing compaction pressure on the compressibility behavior of initially dry compacted samples. A general observation is that the slope of the curves within the respective high-pressure ranges becomes steeper—i.e., increasingly more negative—with decreasing compaction pressure. In relation to the structural models previously discussed, this suggests that for dry-side samples the magnitude and frequency of the large-pore mode decrease with increasing compaction pressure.

Saturated Compressibility

Each saturated-compressibility sample is designated by three letters: The first two characterize the equivalent desired impact level and the desired end-result compaction condition, respectively; the last letter—A, B, or C—characterizes the sustained load during saturation for an equivalent embankment height of 7.8, 15.6, or 23.5 m (25, 50, or 75 ft), respectively. For example, a soaked sample designated by the characters SDA indicates that the sample was initially compacted at an equivalent standard Proctor impact level, to an initially dry-of-optimum condition, and was incrementally loaded and soaked at an applied pressure that corresponds to an embankment height of 7.8 m.

The relations shown in Figure 5 are typical curves for saturated compression versus log time. Each curve has a characteristic type 1 shape, as proposed by Leonards and Girault (9). The division of consolidation into primary and secondary components is illustrated by two techniques: the usual Casagrande approximation and pore-water-pressure measurements. As shown, the amount of compression at the end of primary consolidation (R_{100}), as determined by both methods, is nearly identical. In addition, the time for full dissipation of pore pressure to occur (t_{100} value at $\Delta u = 0$) corresponds well with the Casagrande t_{100} value; however, the former time was typically 15 percent less.

During the service life of an earthen embankment, environmental changes may effect an increase or decrease in the volume of the mass. As a result, the compacted material will undergo changes in void ratio and saturation that may be undesirable in the given application. The one-dimensional percentage volume changes that occurred on incremental loading and wetting are examined below.

Figure 5. Comparison of methods for obtaining R_{100} and t_{100}.

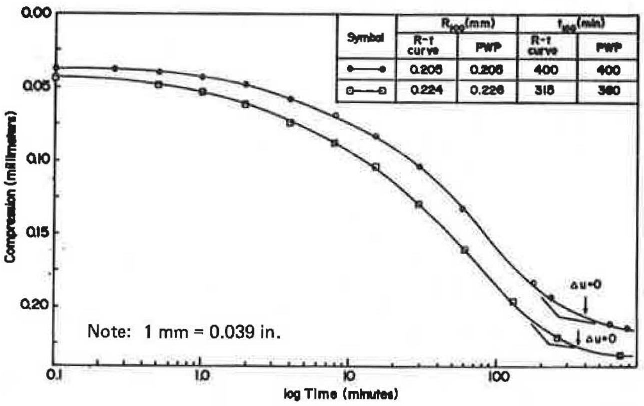
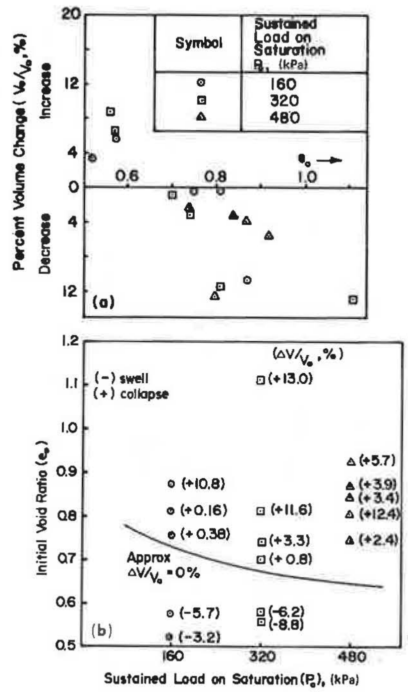


Figure 6. Volume change under load on saturation.



Note: 1 kPa = 0.145 lbf/in^2.

Figure 6(a) shows the relation between percentage volume change on wetting ($\Delta V/V_o$), initial void ratio (e_o), and sustained load on saturation (P_o). [$\Delta V/V_o = \Delta e/(1 + e_o)$, where Δe = change in void ratio on saturation and e_o = initial void ratio.] In general, for a given sustained load, the percentage increase in volume increases with decreasing initial void ratio, and the percentage decrease in volume increases with increasing initial void ratio. Abeyesekera (12) obtained similar results for an isotropically consolidated compacted shale.

To provide a more instructive comparison among these variables, Figure 6(b) shows the same data plotted in a different manner. Observe that, for a P_o equal to 320 kPa (46 lbf/in^2), as the void ratio increases there is less tendency toward volume increase and, once the critical void ratio is reached (value of e_o at $\Delta V/V_o = 0$), there is an increasing tendency toward a decrease in volume. This is similar to the trends stated above. Note, however, that this is not the case for the other two P_o values—160 and 480 kPa (23 and 70 lbf/in^2). That is, the percentage increase and decrease in volume do not occur with decreasing and increasing void ratio, respectively. (Note that an increase in volume is indicated by a negative sign, a decrease by a positive sign. This convention was used throughout because of the limited number of swell data.)

In all cases reported here, the change in volume occurred during the initial wetting period, and little or no change occurred during the incrementing of back pressure. In addition, depending on the initial compaction condition, the volume change was either sudden or time dependent; that is, samples compacted to equivalent low-energy and standard Proctor dry conditions collapsed instantly on wetting, those compacted to equivalent modified dry conditions swelled suddenly on wetting, and those compacted to all other conditions increased or decreased in volume within 1-4 h after wetting.

The collapse phenomenon was first recognized by Jennings and Burland (13). If their explanation is combined with the more recent evidence pertaining to compacted-clay macrostructure (10,14), then the observed collapse phenomenon may be stated as follows.

The dry compacted soil has many large interaggregate pores. On wetting, the surface tension meniscii and flocculated clay buttresses between aggregates are destroyed and become more dispersed, respectively. Consequently, the aggregates will fuse and displace into the available pore spaces. The soil fabric that results from loading, wetting, and subsequent collapse will resemble that of a wet compacted sample—i.e., fewer interaggregate pores and a more dispersed macrostructure.

Similarly, the volume-change behavior of samples compacted to other initial conditions can be explained in terms of compacted-clay structure. Equivalent low-energy and standard Proctor wet-side and at-optimum samples exhibited little volume change on wetting because of the initially more dispersed structure, the smaller number of large pores, and the few developed meniscii. On the other hand, all modified samples swelled on wetting and, according to Hodek (6), this is the result of (a) local softening at aggregate contacts (volume decrease), (b) expansion of the double layer (volume increase), and (c) rearrangement of the soil skeleton (volume decrease). The effect of b outweighs the total effect of a and c.

Statistical Correlations

Predictive models were developed by using the Statistical Package for the Social Sciences (SPSS) procedural programs on Purdue University software (15). The first phase of the isolation process involved plotting the independent variables against the dependent variable. This was facilitated by using the SPSS procedure SCATTERGRAM. If the scatter plots showed a linear relation, the independent variable was considered highly correlated with the dependent variable.

The next step was to isolate a subset of the independent variables that were considered to be highly correlated with the dependent variable so that an optimal expression with as few variables as possible could be established. This was achieved by using the automatic SPSS search procedure STEPWISE. This procedure combines a forward inclusion of independent variables to the model and a backward deletion of independent variables already in the model at each successive step. In addition, it conducts a statistical test to screen out any independent variable that is too highly correlated with the independent variables already in the model.

The final step of the isolation process was to obtain the best-estimated prediction model from the subset of independent variables isolated by the STEPWISE procedure. This was done by using the SPSS procedure REGRESSION developed by Nie and others (15). Various regression equations were obtained by using different combinations. From these, the procedure for selecting the "best" model was based on the following statistical criteria: For the overall multiple regression equation,

1. The coefficient of multiple determination R^2 is greater than 0.65; that is, at least 65 percent of the variation must be explained by the variables included in the model.

2. The coefficient of multiple determination R_a^2 must increase with each additional independent variable entered in the model.
3. The overall F-test at the α = 0.05 significance level must be met (this tests for multiple linearity of the model).

For the partial regression coefficients,

1. The F-test for each partial regression coefficient at the $\alpha = 0.05$ significance level must be met (this tests whether an independent variable should be dropped from the model).
2. The coefficients of partial determination $r_{i \cdot jk}$ are significant.
3. The 95 percent confidence limit for each b_i is small and does not cross zero (this restriction is similar to the first).

For the computed residuals,

1. The scatter plots of the residuals versus the independent variables (s) show normal constancy of variance trends.
2. The residuals are normally distributed random variables; that is, the values of $e_i/\sqrt{MSE}$ (residuals divided by the error root mean square) must range between +3 and -3.

In that case in which all criteria were suitably met by more than one model, the model with the fewest number of variables was selected provided there was no appreciable difference—i.e., less than 5 percent—in either of the R^2 or R_a^2 values.

Based on the foregoing, the following prediction model was selected for (1 - D) percentage volume change on saturation:

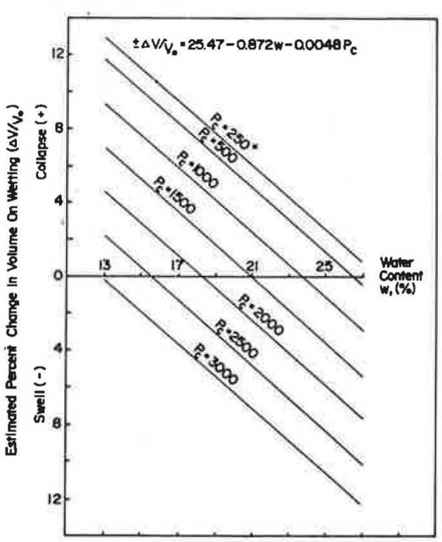
$$\Delta\hat{V}/V_o = 25.47 - 0.872w - 0.0048P_c \quad (1)$$

where

$\Delta\hat{V}/V_0$ = estimated value of (1 - D) percentage volume change on wetting (%),
w = water content (%), and
P_c = nominal compaction pressure (kPa).

Figure 7 shows the relation between the one-dimensional

Figure 7. Prediction of percentage volume change on wetting.



$\Delta\hat{V}/V_0$, as expressed by Equation 1 (P_c values are in kilopascals). Collapse and swell behavior are forecast in this figure by the linear combination of water content and compaction pressure. Moreover, Figure 7 shows that, for a given water content, as the compaction pressure is increased there is less tendency toward collapse (volume decrease) and more toward swell (volume increase). Similar trends are established for increasing water content at a constant compaction pressure.

The effects of water content and compaction pressure on percentage volume change can be outlined as follows:

1. For a given compaction pressure, as the water content decreases, the pore-water pressures become more negative and there are many developed meniscii. On loading and wetting, there is more tendency toward volume decrease.
2. For a given water content, as the compaction pressure increases, the rigidity of the soil skeleton increases. On loading and wetting, there is a softening effect, but the expansion of the double layer results in a volume increase. Again, however, it is believed that both of these factors must be considered jointly when one examines this behavior.

The previous discussion concerning Figure 6 indicated that the relation among initial void ratio e_o, applied pressure P_o, and percentage volume change on wetting $\Delta V/\Delta V_o$ was not well established because of the influence of compactive prestress. In addition, the fact that initial void ratio and confining pressure were not considered statistically significant may also suggest that void ratio and confinement level have little effect on the volume-change-on-wetting behavior of compacted clays, because of the influence of prestress and fabric.

This model cannot be used with confidence until the results are correlated with results for field-compacted samples of the same soil. Clearly, the variables that control laboratory-compacted behavior may be different from those that control field-compacted behavior (3). If these correlations are accomplished, the designer would have a good handle on the in-service compressibility characteristics of the mass. This research is currently in progress.

CONCLUSIONS

This paper has examined the effect of laboratory kneading compaction on the as-compacted and soaked compressibility behavior of a single highly plastic compacted clay (St. Croix). The experimental and statistical results of this study lead to the following important conclusions:

1. A large percentage of as-compacted compression occurs within the first minute of loading. The relations between compression and time indicated similar conditions of fluid continuity, i.e., continuous air voids, for dry and optimum conditions.
2. For an increasing molding water content and equivalent impact level, the as-compacted compressibility behavior was in agreement with the experimental results presented in the literature. It is believed that this behavior can be explained in view of the more recent evidence concerning the macrostructure of compacted clay (10,14,16).
3. For the soaked condition, the Casagrande approximation for determining R_{100} was in agreement with the value obtained from pore-pressure measurements for a load increment ratio of 0.5.
4. The percentage volume change on saturation could not be explained in terms of initial void ratio and the level of confinement on soaking. It is believed that this percentage change is strongly influenced by compactive prestress, although this was not proved by the experimentation.
5. The best predictive model for one-dimensional percentage volume change on saturation was isolated by using the SPSS procedures SCATTERGRAM, STEPWISE, and

REGRESSION. The proposed model was found to be statistically valid and accurate. Equation 1 allows one to predict or control the given response in terms of important compaction variables—namely, water content and compaction pressure.

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Stabilization of a Sanitary Landfill to Support a Highway

RONALD E. SHEURS AND RAJ P. KHERA

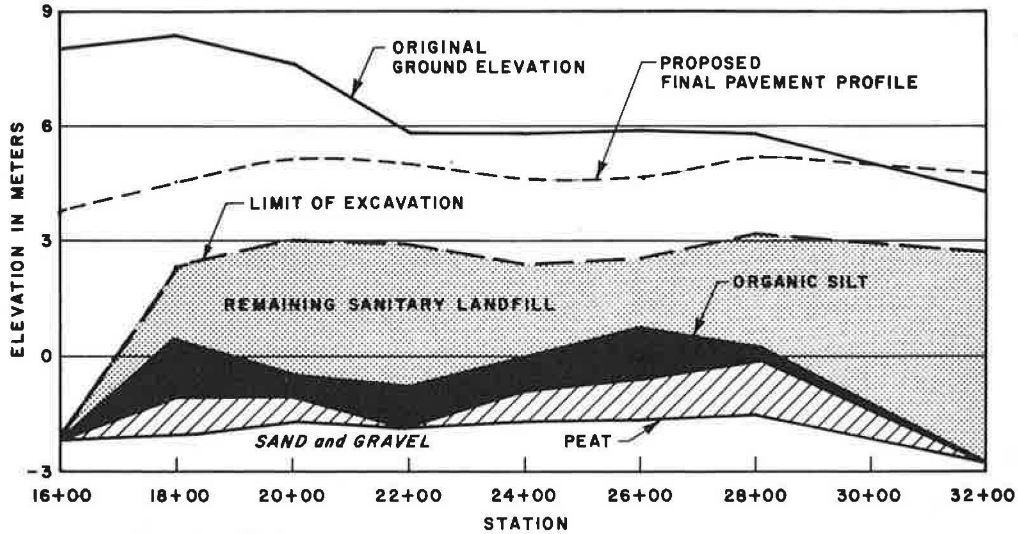
The results of the stabilization of a sanitary landfill by use of surcharges are reported. The New Jersey Department of Transportation has undertaken the construction of two experimental roadways—I-85 eastbound and I-85 westbound—directly over landfills that contain partially decomposed garbage. Field measurements of settlement and pore-water pressure are presented. The data indicate that the settlement response of a landfill area is similar to that of fine-grained soils. Stress history plays an important role in this response. When the ratio between the stress increase caused by surcharge load and the existing stress was <1, the measured strain was only 5-7 percent. When this ratio was 1.4 or greater, the strain varied between 11 and 17 percent. The compression ratio for the sanitary landfill was found to range between 0.16 and 0.20. Piezometric heads were found to be erratic and frequently much higher than the projected values. This was attributed to the expulsion of methane gas from the inner piezometric tube.

Relatively little is known about the behavior of a sanitary landfill subjected to loadings such as buildings or embankments. Extensive foundation problems can result unless the landfill has been stabilized before structures are placed on it. Studies have been reported on a range of stabilization techniques, from chemical injections of grout and fly ash to applications of surcharges (1). Most of the work done to date has been done only under laboratory conditions or small, controlled field conditions.

The New Jersey Department of Transportation (DOT) has undertaken the construction of two roadways directly over landfills that consist of partially decomposed garbage. The project is located on the north side of an active sanitary landfill. The experimental roadways are I-85 eastbound and I-85 westbound. These roadways have a total length of 975 m (3200 ft) and are part of the I-280 construction project in Kearny, New Jersey. Stabilization is being attained by the use of 1.8-m (6-ft) surcharges over a minimum period of 24 months. Settlement plates and piezometers are being used extensively to monitor the progress of the work.

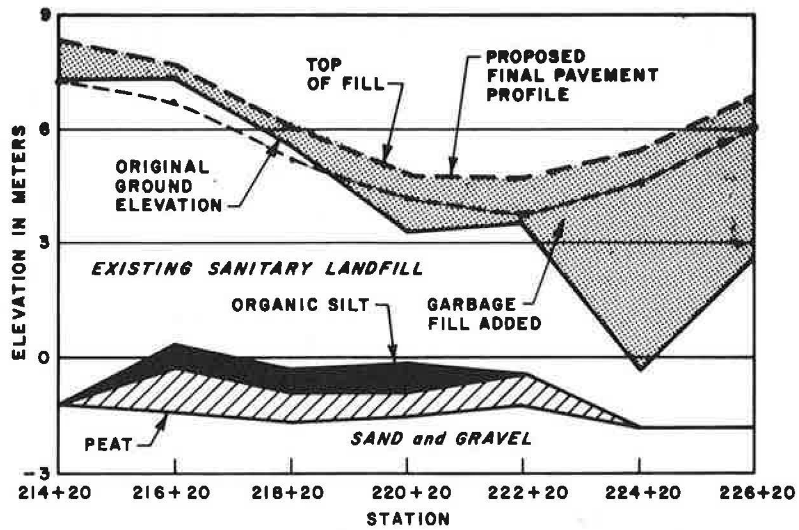
This is an experimental approach to highway construction that the New Jersey DOT has not previously attempted. In the past, it has used overloads with and without the aid of vertical sand drains to accelerate the stabilization of natural deposits of clays and marshlands but not of landfill areas. If adequate stabilization of the

Figure 1. Soil profile for I-85 eastbound.



Note: 1 m = 3.3 ft.

Figure 2. Soil profile for I-85 westbound.



Note: 1 m = 3.3 ft.

sanitary landfill can be achieved in the present project, then a similar or a modified approach can be used on future highway projects.

In April 1977, actual construction and excavation work began at the easterly end of I-85 eastbound, and a general leveling process to a maximum cut of about 9 m (30 ft) began at its westerly end (see Figures 1 and 2). Because of original ground conditions, I-85 westbound required the addition of sanitary landfill. The material used for this fill was obtained from the excavation of I-85 eastbound. All material placed along the westbound roadway was compacted by using from three to five passes of a 312-kN (35-ton) sheepsfoot compactor.

Work progressed at an average excavation of 1913 m^3/day (2500 yd^3/day). This rate decreased considerably following days of rain because of slippery ground conditions and large differential settlements.

A total of 119 360 m^3 (156 000 yd^3) of sanitary landfill was excavated along the experimental roadway. To add to the overall difficulty in excavation operations, approximately 38 260 m^3 (50 000 yd^3) of oil-saturated material was encountered and had to be hauled to specially prepared and lined disposal sites.

SOIL CONDITIONS

Soil borings and laboratory testing of soils were done between 1966 and 1971. The ground elevations and the elevations of the groundwater table recorded on the boring logs and those determined at the time of construction were not in agreement. Most of the landfill material encountered was between 5 and 15 years old. The age of the material was determined from conversations with local authorities responsible for the landfill and the personnel working at the site. This was confirmed from visual examination of the state of the landfill during the excavation of such materials: Newspapers were still readable, cans were not completely rusted, and various types of household refuse were still distinguishable.

The materials in the landfill included normal household refuse, truck bodies, paints, plastics, and chemicals in various states of decomposition, with thin layers of cover material. The underlying material consisted of an average of 1.25 m (4 ft) of gray-black clayey organic silt. Below the organic silt was about 0.9 m (3 ft) of brown peat and fibrous vegetation, followed by 6-7.5 m (20-25 ft) of dense, coarse to fine sand and gravel with traces of silt. The soil profile and the amount of garbage removed and/or replaced along the experimental roadway are shown in Figures 1 and 2.

The soil profile within the experimental sections is basically consistent throughout the project areas, the major difference being that the landfill operations had extended only about one-third the length of the entire project.

Laboratory consolidation tests were performed on undisturbed samples of various underlying natural deposits. Since the number of consolidation tests was limited, based on the nature of the materials present below each of the settlement plates, appropriate test results were selected to best represent the properties of the undisturbed underlying materials, such as organic silt and peat strata. No laboratory consolidation tests were done on landfill materials.

PROPERTIES OF LANDFILL MATERIALS

The density of a typical landfill can vary greatly. The density of the material, as delivered to a sanitary landfill, ranges from 120 to 419 kg/m³ (7.5-26 lb/ft³), and the water content ranges from 10 to 35 percent (2). After the garbage is deposited, it is spread by means of a bulldozer in layers as thick as 3 m (10 ft), which are compacted to different degrees and covered with soil, as required by various local authorities and ordinances. In the past, virtually no control has been exercised in landfill operations, but such operations are being managed better on more recent and better-designed landfills.

The total unit weight of the present landfill materials, which have been in place for several years, was taken as 1129 kg/m³ (70 lb/ft³). Under buoyant conditions, the unit weight was assumed to be 484 kg/m³ (30 lb/ft³). Where the existing landfill was excavated and then compacted (after a certain amount of sorting, which consisted of removal of large articles such as refrigerators, truck bodies, and washing machines), its unit weight was taken as 1450 kg/m³ (90 lb/ft³).

POTENTIAL PROBLEMS IN LANDFILL AREAS

The major areas of concern from a geotechnical viewpoint were the large total and differential settlements, both short- and long-term. The short-term settlements occur during construction as a result of the operation of the construction equipment. Such settlements make the work with construction equipment very difficult, time-consuming, and expensive. The long-term settlements are the result of the weight of the material itself and of applied loads as well as the decomposition of the landfill materials (chemical and biological).

Other problems arise from the potential of spontaneous combustion and possible ill effects on workers attributable to chemical actions and the generation of gases.

SETTLEMENT PLATFORMS AND PIEZOMETERS

All settlement platforms were placed three in a group, and each group was located 61 m (200 ft) on center. At each location, all platforms were placed at the same elevation, one unit at each shoulder and one along the centerline of the proposed finished pavement. The results reported were for those settlement platforms along the centerline of the roadways.

Before the placement of the settlement platforms along I-85 eastbound, a 15- to 30-cm (6- to 12-in) layer of cohesionless material was placed to provide a suitable level base for the settlement platform. The settlement platforms consisted of a 0.9-m by 0.9-m by 12.7-mm (3-ft by 3-ft by 0.5-in) steel plate attached to a 1.22-m by 12.7-mm (4-ft by 0.5-in) standpipe. All plates were set level, and standpipes were set plumb. Base elevations of settlement plates are shown in Figures 3 and 4.

After the settlement platforms had been placed, heavy liquid piezometers were installed so that the piezometer tips were in the underlying natural soil deposits. The piezometers were intended to serve as a control on the rate of placement of surcharge, which was designed to be placed at a weekly rate of 1.22 m (4 ft). The monitoring of the piezometers indicated that the piezometric heads did not conform to the expected values but were highly erratic, as Figure 5 shows. This response was attributed to the expulsion of methane gas from the inner piezometer tube. Thus, the effectiveness of the piezometers became highly questionable and their readings unreliable. So far, however,

Figure 3. Settlements for eastbound roadway.

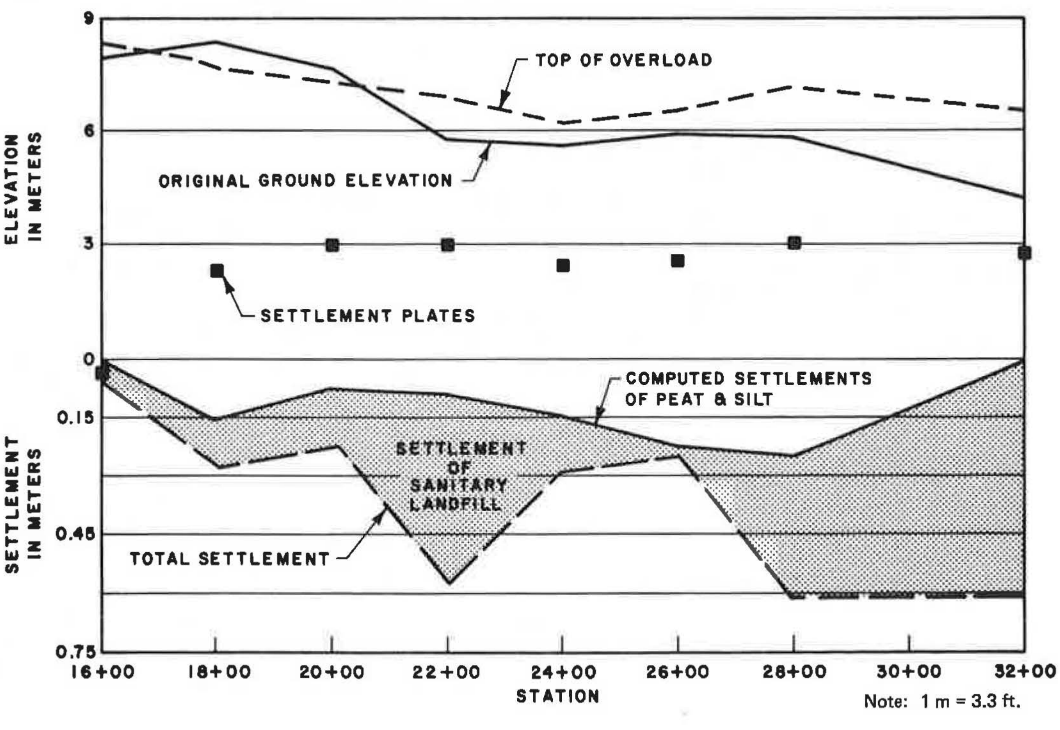


Figure 4. Settlements for westbound roadway.

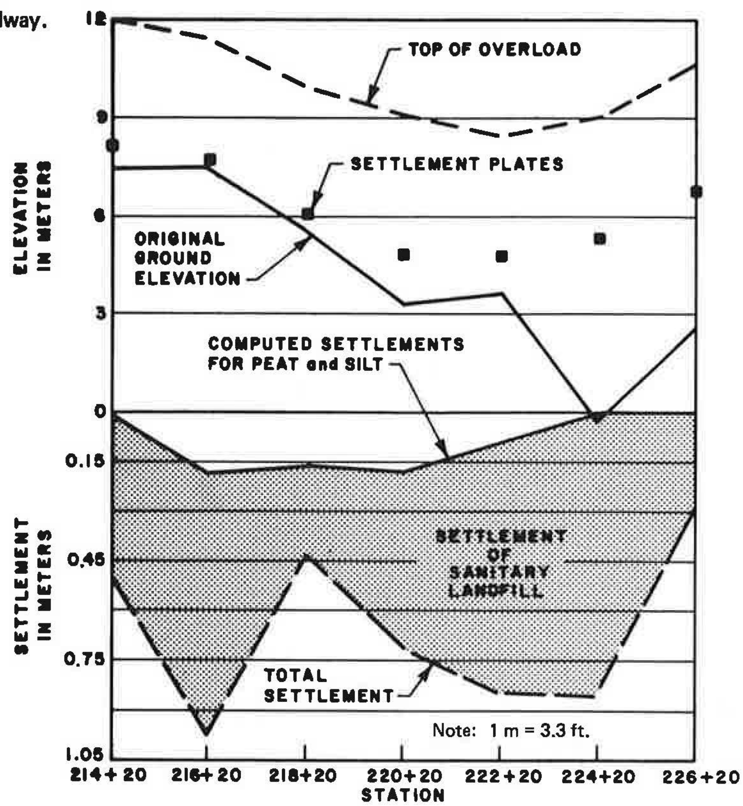
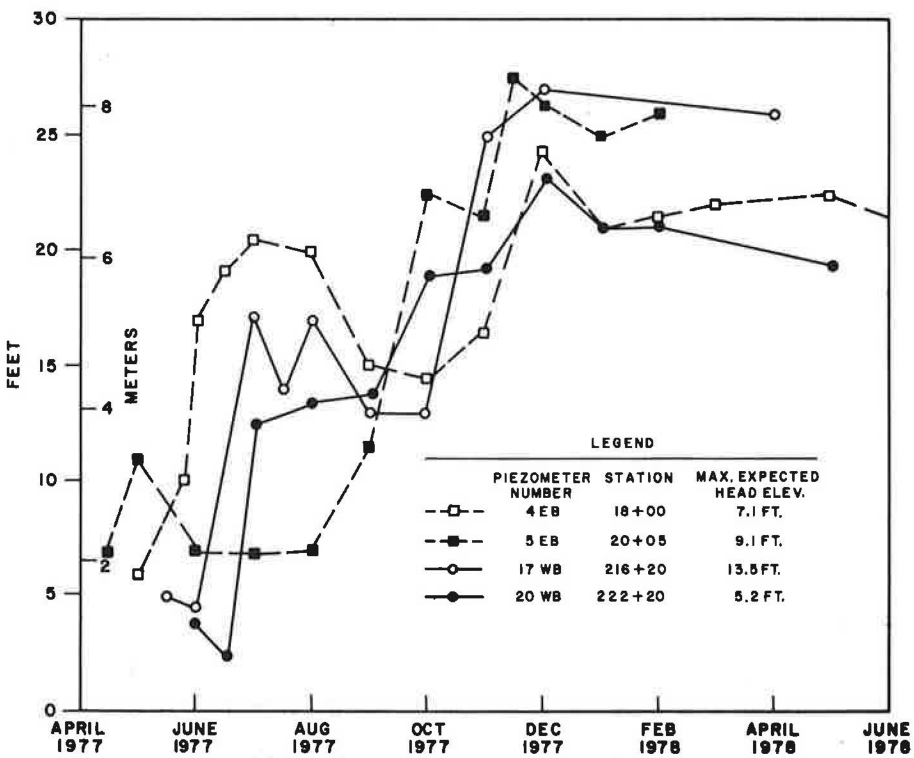


Figure 5. Piezometric heads for both roadways.



failure has not occurred at this rate of application of surcharge.

SETTLEMENT DATA AND ANALYSIS

Settlement-plate readings were taken for all settlement platforms from time to time. Figure 6 shows plots of settlement values for two of the settlement platforms for the eastbound section. Plots for the westbound section are shown in Figure 7. Settlement plots for stations under which only landfill existed but no peat or organic material was encountered are shown in Figure 8.

In general, all of these settlement-time curves are alike and not much different in shape from those observed for fine-grained soils. In the early periods of time, the settlement increases rapidly and then continues to increase

Figure 6. Settlement time for eastbound roadway.

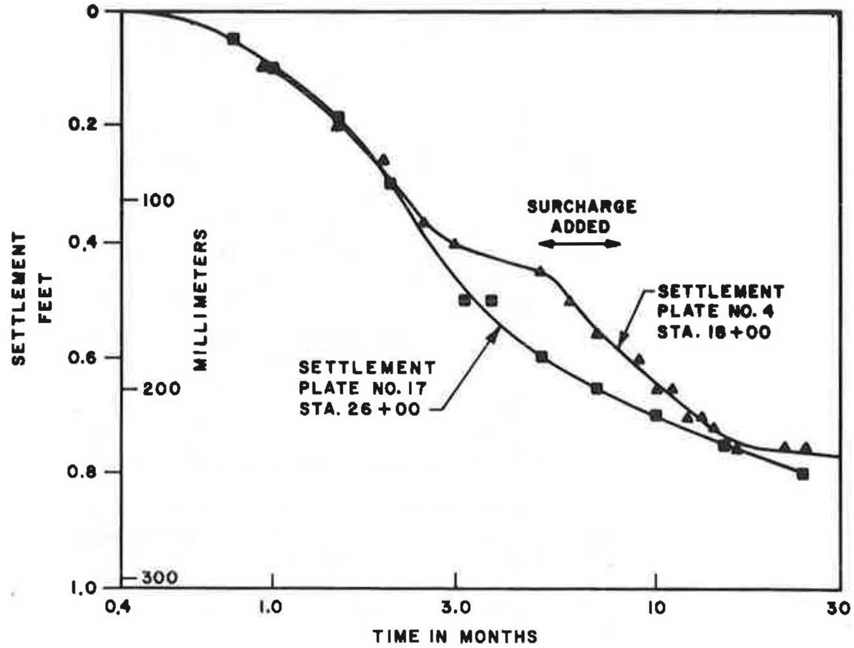
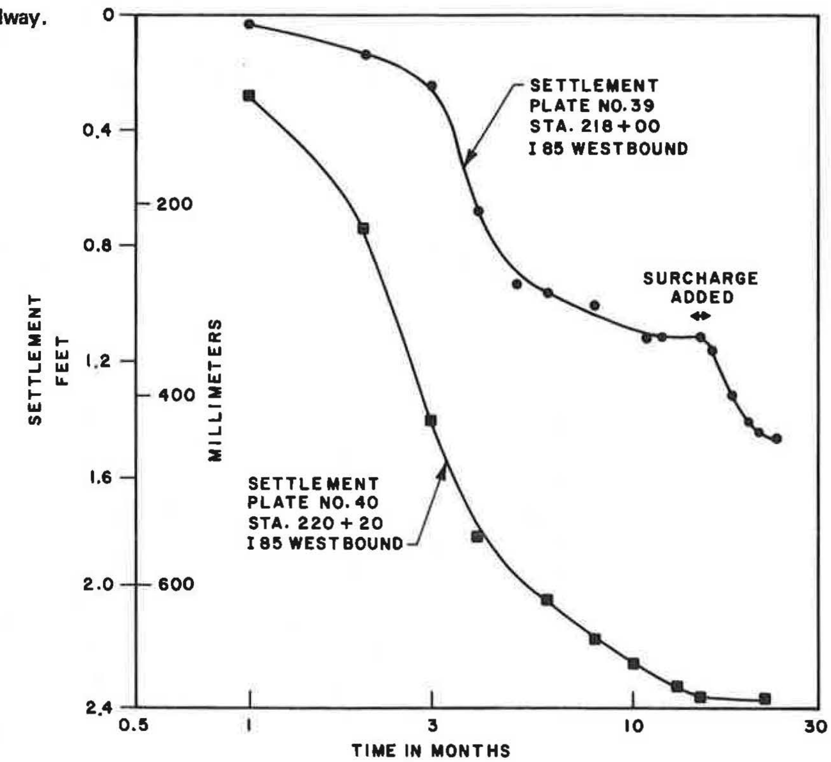


Figure 7. Settlement time for westbound roadway.



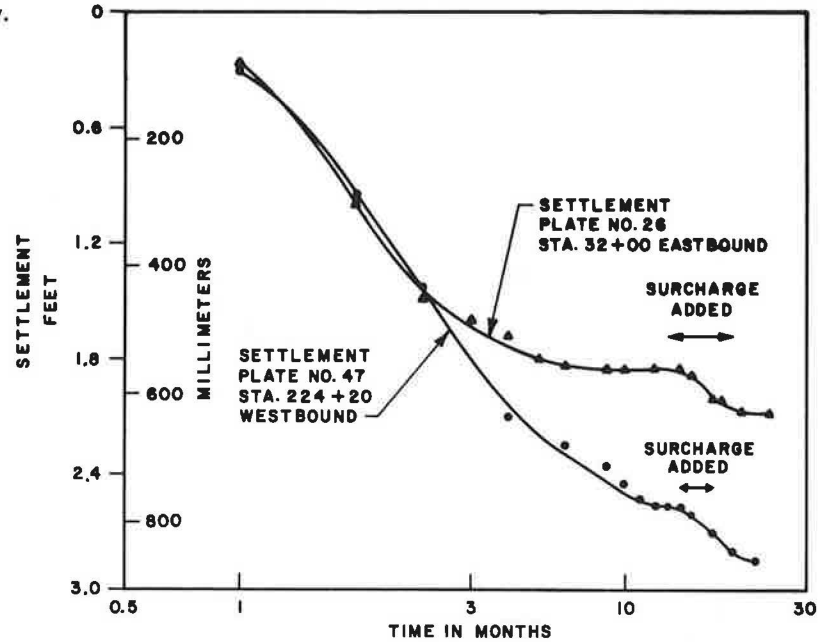
at a decreasing rate. At some stations where additional surcharge was applied at a later date—e.g., SP-26, SP-47, SP-8, and SP-39—as one would expect, the rate of settlement increased.

The thickness of the garbage fill along the westbound roadway varies from 5 to 9.5 m (16.5-31 ft), and the thickness of the underlying peat and organic silt ranges from only 0.75 to 1.7 m (2.5-5.5 ft). Therefore, a large proportion of the total settlement is contributed by the garbage fill. In general, the total strain varies between 11 and 14 percent, with the exception of three stations. The strain values are lower at the two end stations (214+20 and 226+20) because the overload extended only about 15 m (50 ft) beyond the location of the settlement platforms. The low strain value at station 218+00 cannot be explained.

Below the settlement plates where there was only landfill and no peat or organic silt was found, as was the case for station 32+00 eastbound and station 224+20 westbound, the measured strain ranged between 11.4 and 11.8 percent. This is in close agreement with the values recorded for the westbound roadway.

For both the roadways, the total settlement attributable to surcharge varied, depending on the nature of the underlying materials. At the time surcharging was completed, an average of about 66 percent of the total settlement had occurred, whereas the settlements one month and three months after the completion of surcharging were, respectively, 77 and 83 percent of the total

Figure 8. Settlement time for landfill area only.



settlement. Chang and Hannon (4) reported that 80 percent of the settlement was attained within 30 days after the completion of overload on CA-52.

In order to interpret the settlement response of the eastbound roadway, an analysis was carried out on the basis of the amount of landfill material removed or excavated and the amount of surcharge added. In other words, the stress-history effects were considered:

1. Station 16+00 has the lowest values of settlement because there is neither landfill material nor organic silt and peat below it. The measured deflection of 60 mm (0.2 ft) is attributable to the settlement of newly placed sandy backfill material that the contractor had end-dumped into water after having excavated all organic materials down to the sand and gravel stratum.
2. Four of the stations (18+00, 20+00, 24+00, and 26+00) show strains between 5 and 7 percent. For each of these stations, the ratio between the increase in stress attributable to the surcharge load and the stress from the excavated landfill materials was ≤1.
3. The remaining three stations for which the strains vary between 11.4 and 16.8 percent had corresponding stress ratios of 1.4 and greater.

SETTLEMENT OF NATURAL ORGANIC MATERIALS

An attempt was made to separate the settlement contribution of the landfill materials from those of the underlying natural deposits of peat and organic silts. Laboratory test results were used to compute the settlement in the natural deposits. The difference between the settlement-platform readings and those computed for peat and organic silt was considered to reflect the settlement of the sanitary landfill. Since consolidation tests were available for a few typical samples, based on the classification of underlying soils, the most appropriate soil parameters were selected for computation of settlement in the underlying natural deposits. One must keep this fact in mind in viewing these results.

Initial calculations were made by using ground elevations and water levels as indicated by the boring logs. The results of this analysis yielded computed settlements somewhat larger than the measured settlements. Upon reexamination, it was determined that groundwater and ground elevation, as shown on the boring logs, were different from those observed at the time of construction. Comparative values are given below, measured in feet in relation to New Jersey data at mean sea level. Water-table values indicate actual groundwater observed in the field at elevation plus 4.0 (1 ft = 0.3 m):

Station	Original Borings (%) Water Table	Original Borings (%) Ground Elevation	Actual Ground Elevation (ft)
Eastbound			
16+00	27.4	29.6	26.3
18+00	19.3	27.8	27.4
20+00	14.8	25.3	24.8
22+00	13.9	16.9	19.2
24+00	7.3	17.8	18.7
26+00	12.8	17.3	19.2
28+00	11.3	14.3	18.9
32+00	7.7	10.7	14.0
Westbound			
214+20	–	–	24.4
216+20	22.0	23.0	24.8
218+20	11.8	16.3	18.2
220+20	6.5	16.3	11.0
222+20	2.3	11.3	12.2
224+20	5.8	5.8	-1.0
226+20	5.8	5.8	+8.6

The error in the boring logs was believed to be caused by incorrect inferences by the boring crew, either because of perched water conditions or the water used during boring operations. After adjustments were made to ground and groundwater elevations, settlement computations were revised. Both computed and measured settlement values are given in Table 1.

For the westbound roadway, the strain in the garbage fill and the underlying peat and organic silt is about the same. For the eastbound roadway, similar agreement is found for all but two stations.

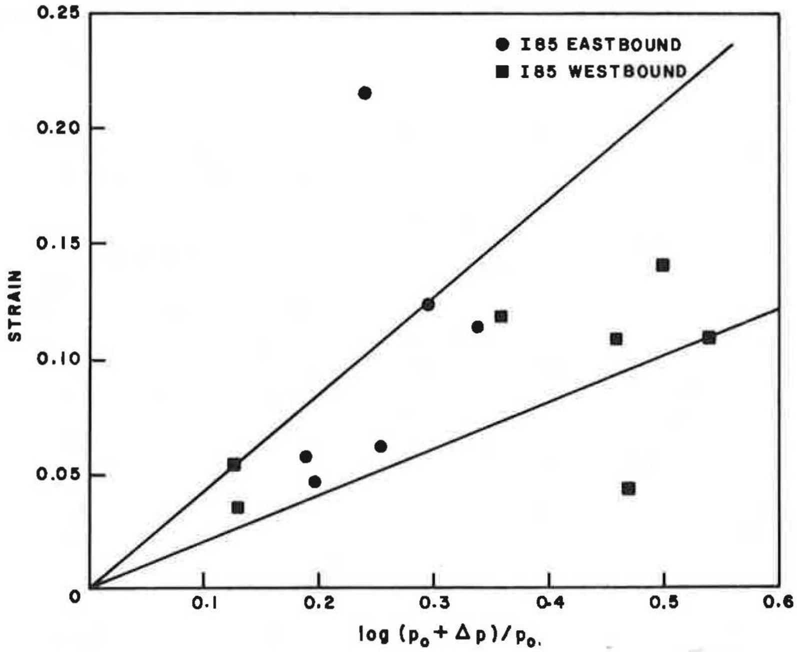
In the case of station 26+00, the computed settlements for the underlying natural deposits were equal to the total measured settlements, which indicates that the landfill stratum that was 1.8 m (6 ft) thick had no settlement. Before the installation of the settlement platform, this area was used as a haul road. This construction activity resulted in a high degree of compaction of the underlying garbage fill prior to the application of the surcharge load.

Table 1. Computed and measured settlement values.

Station	Settlement Platform	Thickness (ft) Landfill	Thickness (ft) Peat and Silt	Settlement (ft) Total (measured)	Settlement (ft) Peat and Silt (computed)	Settlement (ft) Landfill	Strain (%) Total	Strain (%) Landfill
Eastbound								
16+00	2	-	-	0.20	-	-	-	-
18+00	4	6.5	8.0	0.90	0.53	0.37	6.2	5.7
20+00	8	11.5	4.0	0.75	0.22	0.53	4.8	4.6
22+00	11	7.3	3.5	1.85	0.27	1.58	16.8	21.6
24+00	14	8.0	5.5	0.95	0.46	0.49	7.0	6.1
26+00	17	6.0	8.0	0.80	0.80	0.00	5.2	-
28+00	20	9.5	6.0	2.00	0.82	1.18	12.9	12.4
32+00	26	17.5	-	2.00	-	2.00	11.4	11.4
Westbound								
214+20	32	31.0	-	1.65	-	1.65	5.3	5.3
216+20	35	24.0	5.5	3.25	0.63	2.62	11.0	10.9
218+00	39	21.0	4.0	1.45	0.56	0.89	5.8	4.2
220+20	40	16.5	4.5	2.35	0.58	1.77	11.2	10.7
222+20	44	17.5	2.5	2.80	0.34	2.46	14.0	14.1
224+20	47	24.2	-	2.85	-	2.85	11.8	11.8
226+20	50	28.7	-	0.95	-	0.95	3.3	3.3

Note: 1 ft = 0.3 m.

Figure 9. Strain versus stress for both roadways.



At station 22+00, after the adjustment was made to the total settlement, a much greater strain was indicated to have occurred in the landfill. Since most of the rest of the data indicate that the sanitary landfill had a strain of about 11 percent, incorrect choice of the soil parameters for the peat and organic silt may have been one of the reasons for this discrepancy.

SETTLEMENT OF LANDFILL MATERIALS

Along the eastbound roadway, before the placement of settlement platforms, it was necessary to remove about 1.5-6.1 m (5-20 ft) of the landfill material while leaving several feet of the underlying landfill in place. In the removal process, such articles as the bodies of trucks and washers and dryers were uncovered. Along the westbound roadway, the top of the existing garbage fill was below the points where settlement plates were to be installed. The landfill material that was excavated from the eastbound section was cleared of larger articles and recompacted along the westbound section. The compacted landfill material was as deep as 5.5 m (18 ft) in some sections. Because of the sorting and controlled compaction conditions, the added garbage fill was considered to have a higher density [1450 kg/m^3 (90 lb/ft^3)] than the existing fill [1129 kg/m^3 (70 lb/ft^3)], which was neither sorted nor compacted in any controlled manner.

The relation between strain and stress for both roadways is shown in Figure 9. All but two points lie between two lines that have a slope-compression ratio of 0.16 and 0.20. As discussed before, the computed strain value for the point that lies above the upper line that represents SP-11 is most likely incorrect. Another point previously discussed is the fact that the low strain value for SP-39, which lies below the lower line, could not be explained. Thus, disregarding these two points, the average compression ratio of the landfill is 0.18.

CONCLUSIONS

Based on readings taken from several settlement platforms and piezometers along I-85, which is to be constructed on a stabilized sanitary landfill material, the following conclusions can be made:

1. Piezometer readings were marred by the methane

gas that was still being generated in the landfill.

2. Settlement-time curves were found to have shapes similar to those of fine-grained soils.

3. Strain in the sanitary landfill was between 11 and 14 percent for the applied stress range.

4. The underlying deposits of peat and organic silt exhibited strains similar to those of the overlying landfill materials.

5. The compression ratio for the sanitary and fill material, some of which was undisturbed and some of which was excavated and recompacted, was between 0.16 and 0.20 and averaged 0.18.

6. The observed rates of settlement correlate well with those reported for a highway constructed on sanitary landfill in California (4).

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Prediction of Density and Strength for a Laboratory-Compacted Clay

D. W. WEITZEL AND C. W. LOVELL

Predictions of compacted soil strength for a highly plastic clay are developed by statistically correlating the results of unconsolidated-undrained triaxial tests with the compaction variables of water content, dry density, and compactive work. Work was calculated from measurements of the force and displacement of a kneading-compactor foot as it loaded the soil. The statistical models for prediction of density and strength gave results consistent with well-documented experimental evidence. The model for density prediction dry of optimum included variables of water content and a compactive work ratio; for compaction wet of optimum, dry density was a function of water content only. The model for strength prediction dry of optimum included variables of water content, dry density, degree of saturation, and confining pressure. Wet of optimum, the logarithm of strength decreased linearly with initial void ratio. All of these models had good statistical validity. By using such models, the designer can predict soil strength at particular compaction levels or, if a minimum strength value is required, the necessary levels of compaction values can be estimated. These relations were developed from laboratory compaction. Similar relations for field compaction are being developed with the intention of correlating the two.

Compaction is commonly used to improve the strength of embankments, although the placement specification usually controls only certain compaction variables. This study developed predictions of strength from compaction variables that are more commonly and simply measured than strength. The as-compacted strengths of a laboratory-compacted, highly plastic clay were measured in unconsolidated-undrained triaxial tests. Samples were prepared by kneading compaction to densities that fit on three impact-energy curves, on each of which were four water contents. Samples were then sheared at four levels of confining pressure to simulate a variety of compaction conditions and embankment depths. In addition, the work expended to compact the soil was estimated from measurements of force and displacement for the compactor foot.

The results of these tests were used in statistical regression analysis to develop prediction equations for density and as-compacted strength in terms of the compaction variables. Correlation of these results with similar ones currently being developed for field compaction will allow better control of short-term shear behavior in embankments. Such predictions are of particular interest as (a) strength becomes more extensively used as a compaction specification element and (b) greater attention is paid to the potential overloading of the compacted soil by construction equipment.

LITERATURE REVIEW

The clay fabric established by compaction has an important effect on soil behavior. Modern fabric explanations were first introduced by Barden and Sides (1) and then by Hodek (2) and his "deformable aggregate" theory (deformable aggregate is an agglomeration of clay particles, or macropeds).

The size and distribution of the pore space of compacted-clay fabric have also been studied considerably in recent times. Bhasin (3) defined pore-size distributions for several different clays at various energy levels. The distributions of the pore sizes for soil at equal porosities wet and dry of optimum were very different: The dry sample had larger pores than the wet one. In addition, as compactive effort increased at a constant water content dry of optimum, the quantity of larger pores was vastly reduced until a point was reached at which further changes in the pore-size distribution would not occur. These results agreed with those of Sridharan and others (4) and Ahmed and others (5). Garcia-Bengochea (6) also reached similar conclusions for a 50-50 silt-kaolin mixture.

Such studies deemphasize the role of individual clay particles in the compaction process and focus on the nature and action of collections of particles into groups called domains, packets, macropeds, or aggregates. The arrangements of these aggregates vary significantly wet and dry of optimum. Dry of optimum, the aggregates are distinct. The void space is principally between aggregates, and a considerable quantity of it is in larger pores. As optimum water content is approached, the soil gets closer to its plastic limit and the aggregates become more deformable. Hence, in compaction the aggregates distort and squeeze together, and this reduces the number of large pores. Past the optimum water content, the aggregates become much less distinct, and the pore space is essentially in intra-aggregate pores, a much finer range of sizes. At constant water content, increases in energy also change the arrangement of aggregates. As energy increases, aggregates become more broken or deformed, and the quantity of large pores is reduced.

Rutledge (7) presented some of the first comprehensive results of unconsolidated-undrained (U-U) strength testing of compacted clays. He found that the major variables were minor principal stress, dry density, water content, and degree of saturation. The U-U strengths (a) increased as the minor principal stress increased, (b) increased as the dry density increased, and (c) decreased as the water content and saturation increased. The U-U strength increases with the minor principal stress only until the confining pressure becomes high enough for a sample to become fully saturated. This happens when the air in the sample voids is dissolved in the water as a result of increased pressure (8,9).

Samples of similar fabric and initial water content show that strength will increase with an increase in density. In this connection, Leonards (10) found a unique relation between void ratio at failure and compressive strength that was independent of confining pressure, drainage condition, water content, and degree of saturation. At different initial water contents, different compacted fabrics result, and each initial condition would have a different relation between void ratio and strength. DaCruz (11) found the differences could be accounted for by including saturations and void ratio e; i.e., $e\sqrt{S}$. These values at failure plotted versus log $[1/2(\sigma_1 - \sigma_3)_f]$ showed a linear relation for all compacted samples of DaCruz' residual clay.

Some investigators have used statistical techniques to predict shear-strength behavior. Essigmann (12) performed unconfined compression tests on a laboratory-compacted silty clay. From a list of water content, dry density, energy, strength, and combinations thereof, significant variables were selected by an all-possible-regressions analysis. These variables were further combined and analyzed in separate complete regression runs to obtain valid prediction models. Similar regression techniques were used successfully by Scott (13) to formulate prediction equations for dry density and strength. Price (14) developed prediction equations similar to those of Scott and Essigmann but by a somewhat different statistical approach.

EXPERIMENTAL PROCEDURE AND APPARATUS

The soil used in this study was St. Croix clay, a highly plastic residual soil of sandstone and shale origin, taken from a cut area about 6.5 km (4 miles) south of St. Croix, Indiana. Atterberg limits and classification values determined for St. Croix clay are given below:

Category	Classification or Value
Unified soil classification	CH
AASHTO classification	A-7-6 (27)
Atterberg limits (%)	
w_L	52
w_p	23
I_p	29
Clay fraction, < 2 μm (%)	40
Specific gravity (G)	2.79

Kneading compaction was used to prepare samples for triaxial testing. It was felt that this method, among the common laboratory types, was the most like field compaction; i.e., the shearing strains and loading patterns that occur in compaction are more similar to those in the field. Work applied to the soil was measured in terms of the load and displacement of the foot during compaction, for every tamp applied to the soil. Details of the instrumentation are given elsewhere (15). Strength samples were obtained by pushing thin tubes into the compacted sample.

The apparatus for measuring volume change was like that designed by Chan and Duncan (16). It measures the quantity of water that flows into and out of the triaxial cell chamber as a result of changes in the sample volume. Volume changes of 0.01 cm^3 (0.000 61 in) could be detected. Application of the load was at a constant rate of strain of 0.058 mm/min (0.0023 in/min). This rate was chosen to approximate the rate used in the consolidated undrained triaxial testing phase of this project (17). The triaxial test was run until the sample reached its peak compressive strength or 20 percent axial strain, whichever came first.

DISCUSSION OF RESULTS

Testing Program

The results of impact compaction tests on St. Croix clay are shown in Figure 1. The three compaction curves are the results of three different energy levels: modified Proctor (ASTM D1557), standard Proctor (ASTM D698), and low-energy Proctor (a 15-blow "standard Proctor"). Triaxial compression samples were tested at four water-content levels for each energy level. These water contents were chosen at equal saturation levels—70.5, 77.5, 85.0, and 92.5 percent—as shown in Figure 1. For each level of density and water content, samples were tested at confining pressures of 0, 138, 276, and 414 kPa (0, 20, 40, and 60 lbf/in^2). The confining pressure of 414 kPa approximately corresponds to the vertical pressure a sample would experience at an embankment depth of 20 m (70 ft).

To save space in subsequent sample descriptions, a code was adopted. The confining pressure was denoted by the letter C and a number of 0, 1, 2, or 3: C0 for zero confining pressure, C1 = 138 kPa, C2 = 276 kPa, and C3 = 414 kPa. The energy level was represented by the letters L, S, or M, for low-energy Proctor, standard Proctor, and modified Proctor, respectively. A number from one to four accompanies this letter to define the saturation level: 1 = 70.0 percent, 2 = 77.5 percent, 3 = 85.0 percent, and 4 = 92.5 percent. The samples for triaxial testing were actually formed by kneading compaction with selected foot pressures. Hence, the designation of an energy level as modified, standard, or low-energy Proctor means that the sample was compacted at the kneading-compaction pressure required to obtain a sample of selected water content and density on one of the impact curves.

Measurement of Work in Compaction

It is customary to designate the compactive work as a total value—i.e., the work required to operate the compaction device or machine. These values differ from the work required to actually compact the soil, since they include energy lost in the machine operation and work done on the soil that does not produce residual densification. Thus, compactive work equals force times the residual deformation that produces densification of the soil. Measures of foot pressure and total deformation under loading with time are reasonably convenient, even with automatic compaction machines, and were used to represent compactive work in this study. This quantity excludes the energy lost in the compactor operation and in the transfer of energy from the compactor into the soil. It does not exclude the energy expended in a nonuseful fashion to produce elastic deformation of the soil. This work measurement may also include nonuseful energy for samples that are compacted wet of optimum. When samples approach full saturation, vertical foot displacements may result in excessive shear displacements and heaving. The rebound and heaving losses are most significant after a number of tamps have acted to initially densify the layer of soil.

Schematic load and displacement curves for a single

Figure 1. Results of impact compaction.

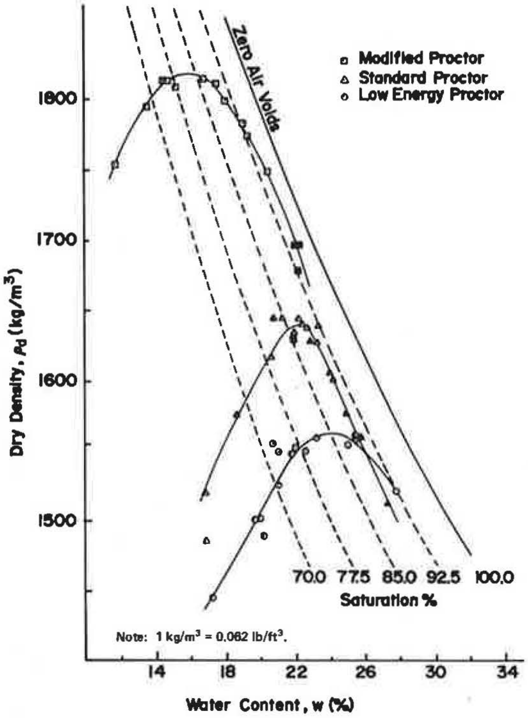
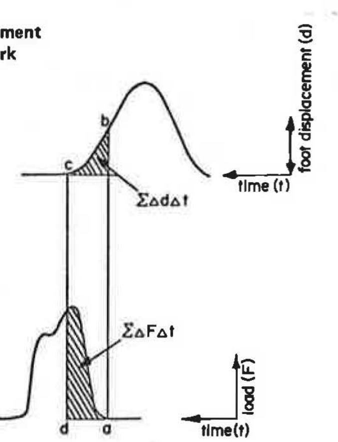


Figure 2. Typical load and displacement curves from recorder output for work measurement.



tamp of the kneading compactor are shown in Figure 2. The upper curve of the plot is the foot displacement with time; the lower curve shows foot loading with time. To determine the work done on the soil, point a was interpreted as the start of loading, and therefore point b was taken as the position of the foot as it contacted the soil. Compression of the soil continued to point c, where no further downward movement of the foot occurred. The force application within the time period a to d was considered to do compactive work. Since loading beyond point d caused no displacement, no further work was done on the soil.

The work done on the soil for this tamp can be expressed in terms of definite increments as

$$(\Sigma\Delta F\Delta t/\Sigma\Delta t) \cdot (\Sigma\Delta d\Delta t/\Sigma\Delta t) = \Sigma\Delta F\Delta d \tag{1}$$

where

t = time,
F = average force, and
d = displacement.

The work calculated in this way for each tamp was summed for all tamps. Although this measure of work is superior to the nominal values ordinarily used, it still contains errors, as indicated above.

The differences between the average work done at two nominal energy levels, at the same saturation level, are given in the following tables (1 kJ/m^3 = 20.9 ft•lbf/ft^3):

Nominal Energy-Saturation Level	Difference Between Average Work (kJ/m^3)
L1,S1	142
L1,M1	1372
L2,S2	224
L2,M2	1443
L3,S3	168
L3,M3	1569
L4,S4	97
L4,M4	1403

Energy Level	Average of Differences (kJ/m^3)
L,S	158
L,M	1447

For example, the difference between the average work of L1 and S1 is 142 kJ/m^3 (2968 ft•lbf/ft^3). These differences are obviously not constant for different levels of saturation, but they are of the same order of magnitude. The average of these differences is shown in part b [e.g., the average difference in work between L and M samples is 1447 kJ/m^3 (30 242 ft• lbf/ft^3)].

The measured work that was done on the soil decreased as water content increased along a curve of constant nominal impact energy (Figure 1). It is not known what compactive work is done in the impact mode. However, the nominal value of foot pressure in kneading compaction follows the same trend as that of the measured work—i.e., it decreases while nominal energy of impact compaction to the same densities remains constant. Hence, it appears that impact is less efficient than kneading compaction for this plastic soil.

Various work and energy ratios are compared below:

Item	Nominal Energy Level L	S	M
Ratio of average work by kneading compaction	1.00	1.88	9.09
Ratio of average kneading foot pressure	1.00	1.60	6.62
Ratio of nominal impact energy	1.00	1.67	7.61

In each case, the low level of work (energy) is taken as unity. The first line represents the measured values of work. The second line represents simply a ratio of the average compactor-foot pressures required to fit the L, S, and M impact curves. The third line contains the nominal energy ratios for impact compaction. Since all of these ratios are about the same, any of them could be used to form a compactive work ratio. The reader is reminded that these ratios show neither the magnitude of the compactive work nor the numerical efficiencies of the compactive modes.

Unconsolidated-Undrained Shear Strength

Failure conditions as a function of energy level and degree of saturation are shown in Figures 3-5. In Figure 3, it can be seen that, for the low-energy Proctor level, strength decreases with increasing water content or saturation. At lower water contents, the pore-water pressures are more negative and so the effective stresses and shear strengths are higher. The samples for the standard and modified energy levels shown in Figures 4 and 5, respectively, show much the same behavior; i.e., as water content increases, strength decreases. However, exceptions are seen in the levels S1 and M1, where, at low confining pressures, the failure lines for low degrees of saturation lie below those for higher degrees. Conlin (18) found similar behavior. The samples at low water content and saturation should have more negative pore-water pressures, but these are effective over lesser areas and the result is lower average effective stress. Application of higher confining pressures compresses the fabric and allows the water to cover more soil area. The result is an increase in effective stress and thus greater shear strength of the soil, as shown by the failure lines of S1 and M1 at greater confining pressures.

If the confining pressure is high enough, decreases in volume during testing may be great enough to cause almost complete saturation. Hence, any further increase in confining pressure increases the pressures in the pore water, but neither the effective stresses nor the shear strength of the soil is changed. This tendency is evident at saturation levels 2 and 3 of the low-energy and standard Proctor failure lines in Figures 3 and 4. The trend of the failure lines for those same saturation levels at the modified Proctor energy level in Figure 5 is not toward a horizontal position. The soil aggregates of the modified Proctor samples are stiffer and stronger (2) and require higher confining pressures to cause significant volume change. The failure lines for saturation level 4 for all energy levels are quite close to the horizontal over the entire range of confining pressures, especially for the low-energy level. Since three samples are almost saturated, confining pressure has little effect on the strength.

Figure 3. q_f versus p_f for low-energy Proctor level showing failure lines.

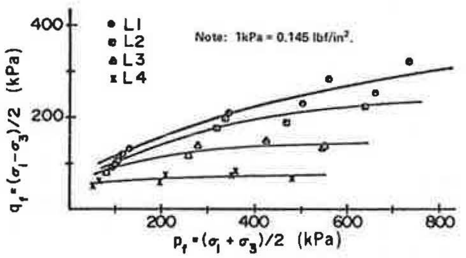
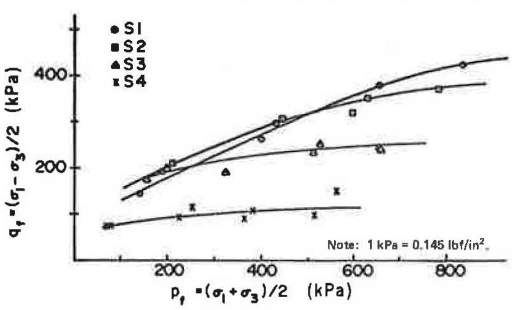


Figure 4. q_f versus p_f for standard Proctor level showing failure lines.



Stress-Strain Behavior

Figure 6 shows stress-strain curves from tests on four low-energy Proctor samples, sheared undrained without confining pressure. The dry-of-optimum samples are stiffer, and the sample at the lowest water content, L1, is quite brittle. This strain behavior is also dependent on the volume changes that occur in the sample. Samples of low water content (dry of optimum), like L1, quickly reach the maximum amount of densification under shear and begin to dilate. Peak load is reached at or shortly after this point. Because the soil aggregates become weaker and more plastic as water content increases from dry of optimum, they yield more before the shear strength is reached. In addition, since wetter samples deform more under the compactor foot, the residual shear stresses are greater and more varied in direction. Greater strain is required to mobilize shear resistance in a given direction. In addition, because the aggregates are more plastic, they will squeeze into a more dense configuration before dilation occurs.

Typical results of low-energy Proctor tests at confinement level 1 [138 kPa (20 lbf/in²)] are shown in Figure 7. All of the curves demonstrate a more plastic behavior, regardless of water content. Application of confining pressure causes considerable volumetric strain and accompanying shear stresses between aggregates. Again, these shear stresses require greater strains to mobilize them all in one direction to failure.

As Figure 8 clearly shows, as confining pressure increases more densification occurs in dry-of-optimum samples. As these volume changes continue to densify the sample, it is able to pick up more load, and thus axial strain also increases. Large confining pressures may result in enough compression to cause the sample to approach

Figure 5. q_f versus p_f for modified Proctor level showing failure lines.

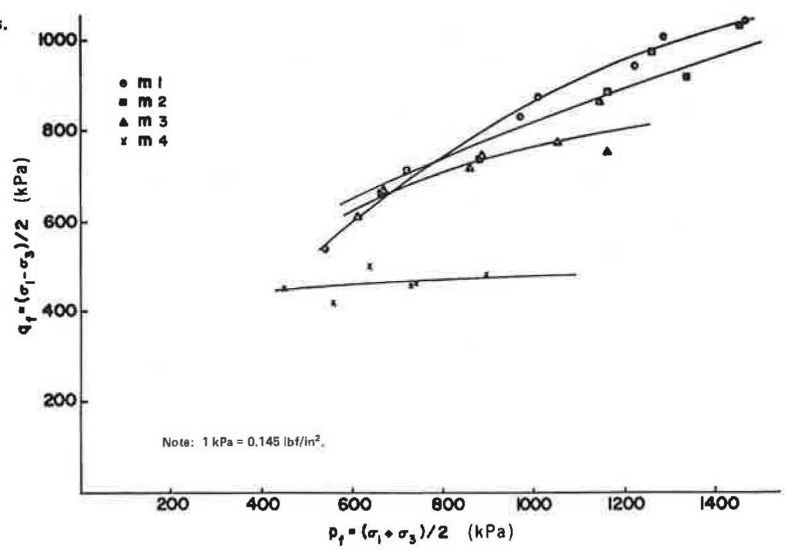


Figure 6. Typical stress-strain curves for tests on St. Croix clay without confinement.

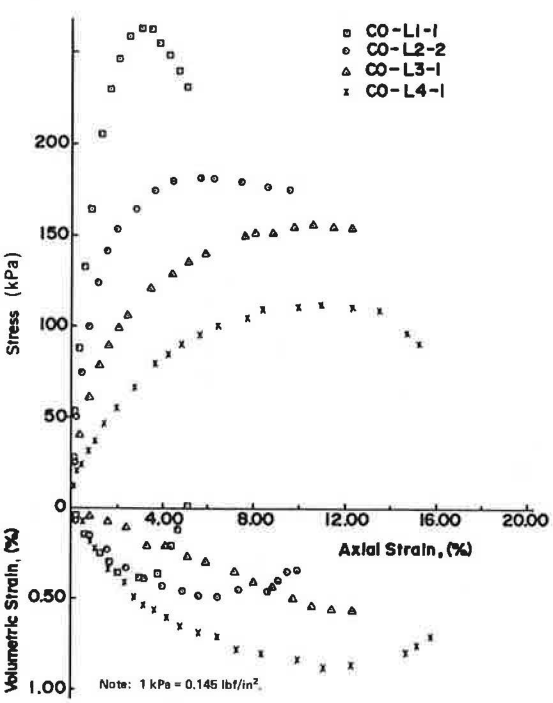
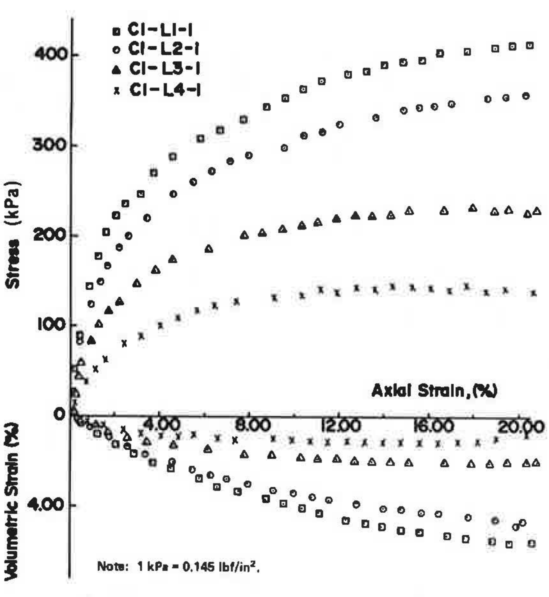


Figure 7. Typical stress-strain curves for tests on St. Croix clay at confinement level 1.



complete saturation and leave less volume change to occur during undrained shear. This may be the explanation for the curves of 414 kPa (60 lbf/in^2) in Figures 8 and 9 crossing those of lower confining pressure. Samples wet of optimum were in all cases close to saturation so that confining pressures had no significant effect on volumetric strain. All samples at the modified Proctor level demonstrated low volumetric strain, regardless of water content and confining pressure (see Figure 10).

Statistical Correlation

The regression procedures and criteria applied in this study

Figure 8. Water content versus volumetric strain at failure: low-energy Proctor level.

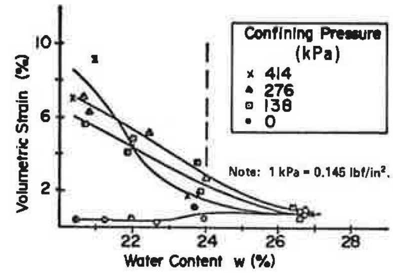


Figure 9. Water content versus volumetric strain at failure: standard Proctor level.

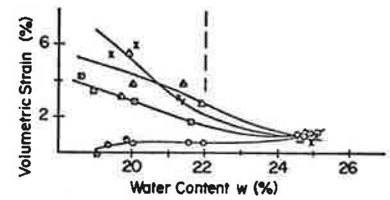
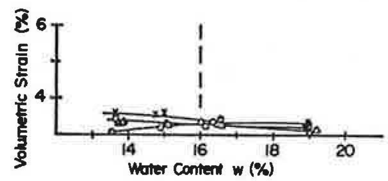


Figure 10. Water content versus volumetric strain at failure: modified Proctor level.



are described in detail by Weitzel (15). The variables used to generate a prediction model for density included water content, compactive work, and the square and square root of both of these. Combinations of all variables in products and quotients were investigated in scattergrams. Also used was the average work ratio, as previously defined, and the deviation of water content from optimum.

It was found that, although overall prediction models for density both wet and dry of optimum were acceptable in most of the statistical criteria, the residuals showed trends that indicated a poor fit to the observed data. This was caused in part by the differences in the effect of compaction work wet and dry of optimum, as described earlier. Therefore, separate models were developed for water content each side of optimum.

The dry-of-optimum model selected for dry density was

$$\hat{\rho}_d = 1338.3 + 1284.0\sqrt{W_R}/w + 0.32\,w^2\sqrt{W_R} \quad (2)$$

where

$\hat{\rho}_d$ = estimated dry density (kg/m^3),
W_R = average work ratio, and
w = water content (%).

It is, of course, important that this equation, and those that follow, not be extrapolated beyond the range for which data were available to develop them. Details on the zones of observed data are given by Weitzel (15).

The prediction model developed for dry density wet of optimum was

$$\hat{\rho}_d = 961.8 + 15\,564.6/w \quad (3)$$

This equation essentially describes a line of constant saturation such as those in Figure 1. By definition, dry density is related to water content and to saturation S by

$$\rho_d = SG\rho_w/(Gw + S) \quad (4)$$

Figure 11. Predicted compressive strength versus dry density at various constant water contents dry of optimum and confining pressures of 0 and 138 kPa.

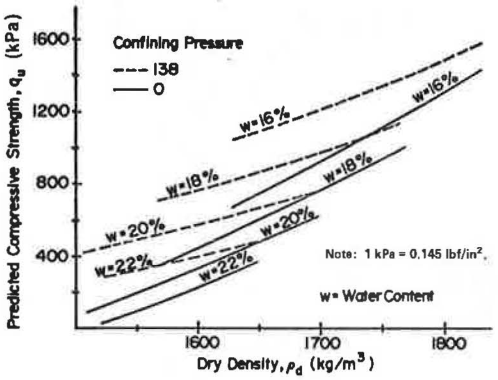
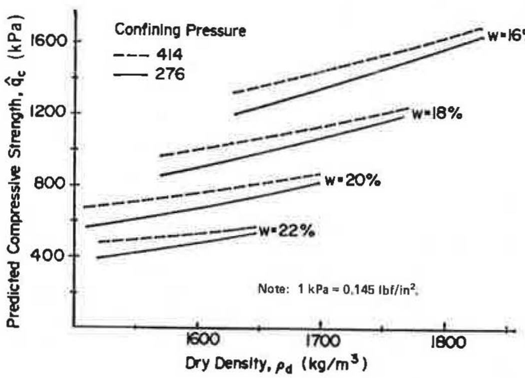


Figure 12. Predicted compressive strength versus dry density at various constant water contents dry of optimum and confining pressures of 276 and 414 kPa.



where ρ_w = density of water and G = specific gravity of solids.

At constant saturation, all of the variables are constant except water content, and density is inversely proportional to the water content. The statistical criteria for the density models are excellent, including an R^2 of 0.99 (15).

The variables tested in prediction models for strength were water content, work, dry density, degree of saturation, void ratio e, and confining pressure (σ_3), as well as the squares and square roots of these. The nearly saturated wet-of-optimum samples increased little in strength with confining pressure, whereas, for reasons already discussed, the dry-of-optimum samples were strongly affected by confining pressure. This made a good prediction model for strength for both wet- and dry-of-optimum conditions unobtainable, as the residual plots for these models show. Hence, separate models for wet and dry of optimum were developed.

The model for strength prediction dry of optimum was

$$\hat{q}_c = -1784.8 + 3.1\,\rho_d\sqrt{S_i}/w + 84.0\,(1 - S_i/100)\sqrt{\sigma_3} \qquad (5)$$

where

$\hat{q}_c$ = estimated compressive strength (kPa),
ρ_d = dry density (kg/m³),
S_i = initial degree of saturation (%), and
σ_3 = confining pressure (kPa).

Since initial void ratio e_i is related to water content by $S_ie_i = wG$, Equation 5 can be written as

$$\hat{q}_c = -1784.8 + 8.5\,\rho_d/e_i\sqrt{S_i} + 84.0(1 - S_i/100)\sqrt{\sigma_3} \qquad (6)$$

The variable $e_i\ \sqrt{S_i}$ showed a linear relation similar to DaCruz' data when plotted against log $(\sigma_1 - \sigma_3)_f/2$ (15). Since

$$\rho_d = G\rho_w/(1 + e_i) \qquad (7)$$

the variable $\rho_d/e_i\ \sqrt{S_i}$ becomes a higher-order term that better linearizes the data than the logarithm. In addition, the effect of confining pressure had to be accounted for. The variable that was found to be best for this was the term $(1 - S_i/100)\sqrt{\sigma_3}$. As confining pressure increases, its effect on strength decreases (this behavior was evident in the q_f-versus-p_f plots in Figures 3-5). The factor $(1 - S_i/100)$ reduces the influence of confining pressure as saturation increases.

Predicted compressive strength versus dry density at both constant water content and constant confining pressure for dry-of-optimum conditions is shown in Figures 11 and 12. These figures show that, as density increases and/or water content decreases, so does the strength. However, as confining pressure increases, the volume changes increase so that initial density has less effect on strength. The trend of the curves at zero confining pressure is very similar to the data of Seed and Monismith (19). The best prediction equation for compressive strength of samples wet of optimum was

$$\text{Log}(\hat{q}_c) = 1.70/e_i \qquad (8)$$

The statistical criteria for these strength equations are excellent ($R^2 \geq 0.98$). The equations can be adapted for quick and simple use by developing them into charts or nomographs (15).

Such prediction equations can aid in embankment design. For a particular water content and work level, the compacted density that will be obtained in compaction can be predicted. The use of this predicted density, or an observed value, permits the soil strength to be estimated and the stability of the side slopes to be calculated. Conversely, one could start with a minimum factor against slope instability and successively "back figure" to estimate the compaction variable values that would produce it.

However, the limitations on the use of laboratory-compacted samples to predict field-compacted conditions should be recognized. Work is currently under way at Purdue University to correlate the results for laboratory-compacted samples with results for samples compacted in the field.

CONCLUSIONS

As a result of unconsolidated-undrained triaxial tests performed on samples of laboratory-compacted, highly plastic St. Croix clay, the following conclusions are drawn.

1. To fit kneading-compaction results to compaction curves of constant Proctor (impact) energy required less nominal energy in the kneading mode as water content increased. This indicated that the kneading method is more efficient than the impact method.
2. The values of calculated compactive work in kneading compaction showed the same trend as the nominal kneading values.
3. When the measured values of work were compared for different fitted Proctor energy levels at constant saturation, the result was a constant ratio. These work ratios are of the same magnitude as the ratios of nominal impact energy.
4. The behavior of St. Croix clay in unconsolidated-undrained triaxial tests is like that well-established in the literature (7-9, 18). The values of strength, determined at maximum stress difference or 20 percent axial strain, decreased with water content,

increased with density, and increased with confining pressure until the sample reached near-saturation.

5. The volume changes that occur during shear are significant, especially for samples of low water content and density. On the dry side of optimum, volumetric strains during shear increased with increased confining pressure and decreased with increased water content. Volumetric strains during shear for samples wet of optimum were independent of confining pressure.

6. The prediction equations for dry density and strength for the range of variables investigated were shown as Equations 2, 3, 5, and 8 in the text. These relations are in general agreement with the reported literature (10, 19).

7. All developed prediction equations had good to excellent statistical characteristics.

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