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DESIGN AND ECONOMICS OF BITUMINOUS TREATED BASES IN TEXAS

by

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Research Report 14-1F
Bituminous Treated Bases - An Exploratory Study
Research Study No. 2-8-73-14

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PREFACE

This report is issued under Research Study 2-8-73-14, "Bituminous Treated Bases - An Exploratory Study" and presents a review of the performance and economics of bituminous treated bases in Texas. Project 2-8-74-41 has been initiated as a result of this limited type B study.

DISCLAIMER

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The authors wish to express their appreciation to Texas Highway Department personnel in Districts 5, 13, 15, 17 and 25 as well as representatives from divisions D-6, D-8, D-10 and D-18 for their time and efforts expended on defining the current design, construction, and cost information associated with selected pavement sections.

ABSTRACT

Types of tests, test criteria and types of materials suitable for bituminous stabilization have been defined. A review of layer equivalency is included as well as current cost data for both stabilized and unstabilized base courses.

KEY WORDS

BITUMINOUS STABILIZATION, BLACK BASE, PAVEMENT DESIGN

SUMMARY

The cost of bituminous stabilized materials has increased considerably in the last twelve months and is expected to continue upward in the near future. Materials and techniques must be defined that slow down this trend. To these ends the report defines the types of materials that can be utilized for bituminous stabilization and the types of tests and their associated criteria that are utilized to design bituminous mixtures.

A spectrum of material properties will result when these acceptable materials are utilized. The thickness and cost of these alternate materials must be considered such that for given situations performance is equal. It is on this basis that the decision as to what material to be used for a particular pavement layer must be made. Although a variety of material properties must be considered by the engineer, the fatigue, durability and rheologic properties appear to be the most important for bituminous stabilized materials. These properties must be adequately defined in order that alternate pavements can be defined and comparisons made on an economic basis.

Prior to the development of more rational criteria, consideration should be given to adopting layer equivalencies based on literature and data cited in this report. Test methods suitable for both mixture design and pavement design purposes should be developed which will allow for adequate determination of these equivalencies.

Alternate supplies of aggregate materials should be located and their properties defined. New mixing, transporting and laydown equipment should be utilized as it proves effective.

IMPLEMENTATION STATEMENT

Material is included in the report which allows the engineer to determine the types of materials that can be utilized for bituminous stabilized layers. Current test methods and test criteria are reviewed which allow for determination of bitumen contents. Layer equivalencies and cost data are included for typical types of bituminous stabilization. Use of the above information will provide more economical bituminous treated base courses.

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INTRODUCTION

The shortage of high quality aggregates together with increased traffic has created a need for treating local materials for use as base courses. Asphalt has become a common base stabilizer in the last eight years; however, the criteria developed for materials selection and design and construction techniques have been based mainly on requirements developed for asphalt concrete surface courses. Thus, because of these sometimes "strict" requirements, materials and construction techniques are being utilized which significantly increase cost and provide a stabilized material whose properties are in excess of those required by traffic and the environment.

To provide an economical material to satisfy the particular requirements of asphalt base courses, current material selection criteria, construction techniques and pavement design methods should be investigated and altered as necessary.

In 1972 the Texas Highway Department established a type B research study with the Texas Transportation Institute. Project 2-8-73-14 titled "Bituminous Treated Bases - An Exploratory Study" had a study objective to explore the feasibility of developing a more economical asphalt treated base course by investigating new construction techniques and more realistic criteria for materials and design which will provide the desired performance.

The approach utilized to fulfill the study objective included information gathering by a review of the literature, conferences with Texas Highway Department district and division personnel, and by visits to equipment manufacturers and contractors. Information gathered from these sources is presented below. Items discussed include a discussion of desirable mixture characteristics, existing methods of tests and test criteria base course temperatures, layer equivalencies and the types of materials suitable for asphalt stabilization.

DESIRABLE CHARACTERISTICS OF A BITUMINOUS STABILIZED MIXTURE

The engineer is faced with providing a bituminous stabilized mixture to satisfy the needs of a particular situation. Certainly these demands vary from construction project to construction project and are dependent upon such factors as environment, loading conditions and locations within the structural pavement section, among others. In an attempt to consider these factors the engineer must consider the following mixture characteristics and their relative importance for a particular utilization of the bituminous stabilized soil:

1. stability
2. durability
3. fatigue behavior
4. tensile behavior
5. flexibility
6. workability

Few tests have been developed to indicate the flexibility and workability of bituminous stabilized materials. Elongation and certain tensile tests are attempts to measure flexibility while gradation limits and compaction tests have been utilized to control workability.

Tensile tests on bituminous stabilized paving mixtures have been summarized by Heukelom (1). Tests utilized include direct tension, indirect tension dumbbell and "dornprobe" tests. From a review of test data presented by Huekelom the engineer can determine the tensile strength and strain at failure. More recent tensile testing of bituminous stabilized materials has been performed at the University of Texas (2), University of Alberta (3) and the University of California (4).

Fatigue testing of bituminous stabilized materials has been reviewed by Epps and Monismith (56) and Pell (67). These reviews indicate the relative importance of asphalt type, aggregate gradation, aggregate type, air void content and other mixture variables.

Specifications and criteria for bituminous stabilized soils are almost exclusively based on stability, durability and gradation requirements. A survey of state practices has been recently published by the Transportation Research Board (8). This survey indicates that the most widely used stability tests are the Hveem (9, 10), Marshall (9), 11, 12), and unconfined compression (9, 13, 14, 15, 16, 17) tests. Other tests used for stability type determinations include Hubbard-Field (13), triaxial compression (13), repeated load triaxial (18, 19, 20), California "R" Value (18, 19, 21) and various penetration type tests including the California Bearing Ratio (22), the Iowa Bearing Value (23) and Florida Bearing Value (24).

Durability tests which have been utilized for control of bituminous stabilized mixtures include the California Moisture Vapor Susceptibility test (25), immersion compression test (26) and the swell test (27).

Criteria based on these tests have developed. Unfortunately most criteria are based on the suitability of the type for determining its adequacy as a surface material whereas most bituminous stabilized soils are presently being utilized as base or subbase courses. A more nearly adequate criteria should be developed for utilization of this material in the main pavement section as well as the surface course.

In addition to the test criteria being developed for surface course applications the engineer should recognize that the majority of testing has been performed on graded aggregate systems rather than the finer "soil type" materials. This is mainly a result of the increase use of graded aggregate as the aggregate fraction of bituminous stabilized soils (Table 1) (26).

DESIRABLE CHARACTERISTICS OF A MIXTURE EVALUATION PROCEDURE

As discussed above a number of mixture characteristics must be considered to properly evaluate its suitability. Ideally a single test would provide sufficient information to adequately define the mixtures stability, durability, flexibility, fatigue behavior, tensile behavior and workability; however, such a test has not been developed nor is there hope for such a test in the near future. Thus we must consider a number of tests to satisfy our need to define mixture characteristics.

Test geometry and loading conditions of the ideal test must be such that they nearly represent the state of loading encountered in the field by the mixture. Certainly the state of stress in the field is biaxial if not triaxial while the load is repeated and of varying magnitude and duration. Research has indicated that a testing apparatus to perform such a test and the theory necessary to interpret such test results is highly complex and in the near future will not lend itself to everyday use. Thus the engineer has utilized less complex tests and have correlated these results with in-service performance. Re-utilization of this less complex test allows its use for construction control as well as for mixture evaluation.

Basically the engineer would prefer a test to be suitable for construction control and mixture evaluation as described above as well as for utilization in pavement design procedures to determine layer thicknesses. Often these requirements are not compatible. For example, the Hveem stability test can be used for construction control and mixture evaluation but does not provide data suitable for pavement thickness design purposes.

Other basic requirements of the suitability of a test method is that it must adequately delineate between an acceptable and unacceptable mixture. Often test results are expressed as a single number. The range of results of this number must be such that acceptable mixtures can be adequately recognized. In addition the maximum value obtained in a test should not be limited to a specific maximum value.

Methods of laboratory specimen preparation should be such that it approximates field preparation. Mixing and compaction procedures should be carefully controlled and should also closely approximate methods utilized in the field. For example, the use of gyratory and kneading laboratory compaction more accurately represents field conditions than static and impact compaction.

As described above desirable characteristics of a mixture evaluation procedure include but are not necessarily limited to the following:

1. the test should be suitable to define as many mixture properties as possible,
2. test geometry, loading conditions and specimen preparation should represent actual field conditions accurately,
3. the test should be simple, easy to perform and the results should be easily interpreted,
4. the test should be suitable for construction control, mixture evaluation and pavement design, and
5. the test should adequately delineate between acceptable and unacceptable mixtures.

Discussions of tests in current use and their adequacy in the light of criteria presented above will be discussed in the next section.

CURRENT TEST METHODS

Methods of tests currently utilized by state highway departments, county, and state agencies as well as several foreign countries are presented below. These tests are separated into stability and durability tests. Stability tests include the

Hubbard-Field test, the Hveem stabilometer, the Marshall stability, the unconfined compression test, the triaxial compression test and certain penetration type tests. Durability tests discussed include the "Moisture Vapor Susceptibility" test and the "Immersion Compression" tests. Gradation requirements are discussed in another section of the report which discusses the types of soils that are suitable for asphalt stabilization.

STABILITY TESTS

Hubbard-Field Test (AASHTO T 169, ASTM 1138) (28, 29). The Hubbard-Field stability test was developed in the mid-1920's for evaluating the mechanical properties of sheet-asphalt paving mixtures under traffic conditions which consisted of steel wheel wagons. At that time, the test consisted of forcing a specimen 2 inches in diameter by 1 inch in thickness through a 1.75-inch orifice. The punching shear failure closely duplicating traffic conditions of that period. The specimens were compacted and tested at 140°F. If the specimen required a force of 2000 lbs or greater for failure, it was suitable for field use. The test apparatus was later modified to permit evaluation of bituminous mixtures containing coarse aggregate and to act as a durability test as described below.

A coarse aggregate bituminous mixture is placed in a mold 6 inches in diameter and 3 inches in depth. Compaction is effected by a double plunger device under a total load of 10,000 pounds. Prior to testing, the specimen is placed in a 140°F water bath for a minimum of one hour. It is then placed in the testing apparatus and load is applied at the rate of 2.4 inches per minute until failure occurs. Optimum asphalt content is determined by comparing test results with empirical design criteria based upon field performance.

Soil bituminous mixtures utilizing finer aggregate particles can be tested utilizing 2-inch diameter by 2-inch high specimens. Proper curing procedures must be utilized when liquid asphalts are utilized.

A water absorption and expansion test can be conducted on a series of these specimens (ASTM D 915). The specimens are placed in a humid room and partially immersed in water for seven days. The absorption and expansion is then calculated based upon weight and volume increase.

Soaked specimens and specimens which were not soaked are then tested using the Hubbard-Field apparatus. Relative extrusion values of 1000 lb before absorption and 400 lb after absorption are considered minimum for satisfactory field results. Expansion is limited to 5% maximum and absorption to 7% maximum.

Hveem Stabilometer (ASTM D 1560). Estimated bitumen contents can be obtained by use of the Centrifuge Kerosene Equivalent Test. Mechanical kneading compaction is utilized to compact a four-inch diameter by 2.5-inch specimen. The specimen is placed in a Hveem stabilometer, a triaxial type device, and either its stability or "R" value is determined. The axial loading rate is 0.05 inches per minute and the test is usually performed at 140°F although other test temperatures including 100°F and 75°F have been used. A swell test and moisture vapour susceptibility test can also be performed on molded samples.

Marshall Stability (ASTM D 1559). During World War II, the Army Corps of Engineers adopted the Marshall method for mix design and field control of pavements for military airfields and roads. The method was adopted because of its simplicity and suitability for use in the field. Since that time the military has made extensive use of the Marshall method accumulating a large background of experience world-wide.

Specimens are prepared using a free fall hammer for compaction. The specimens are compacted on both sides in a four-inch diameter mold to a height of 2.5 inches. The specimen is placed on its side in a testing apparatus and the load is applied at a rate of 2 inches per minute. The test is performed at 140°F simulating the maximum temperature in the pavement system. The load required to produce failure is the stability of the soil asphalt system. The deformation of the specimen is the flow. These values along with percent air voids and the percent voids in the mineral aggregate filled with asphalt are used in establishing the optimum asphalt content. The properties of the optimum soil asphalt system are then compared with established criteria based upon field performance of control test sections for acceptability.

A durability test utilizing the Marshall apparatus has also been utilized by some agencies (29).

<u>Unconfined Compression Test</u> (AASHTO T 167, ASTM D 1074). A specimen four inches in diameter and 4 inches in height is compacted by the double plunger method and tested in compression at a rate of 0.2 inches per minute. The load at failure is the unconfined compressive strength. One-half of this value is the shear strength while the axial strain at failure can be determined if deformation reading were obtained.

The unconfined compression test is a very simple test which can be accomplished rapidly. Factors such as specimen seating, rate of loading, creep, compaction, curing time, testing temperature have profound effects on results and must be carefully controlled when the test is used for evaluation of soil asphalt. Durability tests utilizing this testing technique have been developed (ASTM D 1075) (29).

Triaxial Tests. Empirical procedures have been used fairly successfully over the years in the evaluation of soil-asphalt mixtures. Yet they have a serious shortcoming in that they are applicable within limits set by laboratory and field correlations. In view of this limitation, a more rational approach to evaluation has been sought through the years. The triaxial test is a step in this direction, in that it permits combinations of three dimensional stress which more closely duplicates the stress in the pavement system. It also measures the fundamental strength parameters of the soil asphalt system, internal friction and cohesion between which valid mathematical relationships exist. For example, Smith (30) related values of cohesion and internal friction to bituminous surfacing mixtures which had been proven stable in the field.

Although several methods have been utilized, specimens are usually compacted in molds up to six inches in diameter with a 2:1 height to diameter ratio. The specimens are placed in a triaxial apparatus and the test conducted either by applying a constant confining pressure and increasing the vertical load to failure; or applying a vertical load and measuring the lateral pressure. Two or more specimens must be tested at different confining pressure in order that a Mohr's rupture envelope can be plotted from which the cohesion and internal friction can be plotted.

Repeated Load Triaxial. The loading condition used in all laboratory evaluation procedures discussed earlier consists of a static mode. The static mode is not representative of moving traffic which a pavement encounters in service. The repeated load triaxial compression test attempts to duplicate the loading conditions representative of moving traffic by providing large numbers of stress repetitions to the specimens having a lateral confinement. Various loading rates are used with a common one being 20 applications per minute with a load duration of 0.1 second. Specimens are often tested at 68°F with confining pressures ranging from 5 to 40 psi and deviator stresses ranging from 5 to 30 psi. A resilient modulus is determined from deviator stress and recoverable axial strain which is used in evaluating the effect of confining pressure, applied vertical stress, curing before and after compaction, temperature, etc. (31).

Most of the reported experience with the repeated load triaxial test has been in the evaluation of asphalt treated base course materials. Terrel and Monismith (31) report success in using the test in measuring the resilient behavior of asphalt treated aggregates. The test is reportedly versatile in that it can be used with all types of paving materials and laboratory prepared or cored specimens from pavement sections can be utilized for laboratory measurements.

California Bearing Ratio (22). The California Bearing Ratio test originally developed for soil testing has been utilized for testing stabilized soils. Impact compaction is utilized to compact 6-inch diameter specimens. A two-inch diameter piston is seated on the specimen and loaded at a rate of 0.05 inches of penetration per minute. The CBR value is the ratio of the load expressed in percent required to cause the piston to penetrate 0.1 inch in the specimen to the load required to cause the piston to penetrate 0.1 inch in a well-graded crushed stone. The value has been correlated to field performance of pavement sections and can be used for thickness analysis.

Iowa Bearing Value (23). The Iowa Bearing Value test was developed as a substitute for the California Bearing Ratio Test primarily to reduce laboratory testing time. It is used predominantly for fine grained soils. A 2-inch diameter by 2-inch specimen is used with a 5/8 inch penetration rod. The rod is loaded so as to produce a penetration rate of 0.05 inches/minute. The test results have been correlated with CBR.

Florida Bearing Value (24). Rate of loading and method of compaction standards were not established in the original Florida Bearing Value test, this reproducibility was difficult. In the Modified Florida Bearing Value test, these standards were established and better results were obtained. Asphalt content was established on the basis of grain size distribution. The mixture is tamped in a 4-inch diameter by 3-inch mold and compressed with a 25,000 lb load. The specimen is heated to 140° F and tested. Testing consists of loading a cylindrical rod (1 sq. in. area) at the rate of 92 lb/min. until a load of 60 lbs. is reached. The specimen carries the 60 lb load for two minutes. The load is increased in 10-pound intervals with 2-minute static loads until failure occurs. The Modified Florida Bearing Value is the maximum load before failure.

DURABILITY TESTS. Water absorption into soil asphalt mixtures results in expansion and loss of stability. The swelling is nearly proportional to the amount of water absorbed; however, its affect varies depending upon the soil and type treatment. Swelling often results in pavement failures due to distortion of the pavement surfacing. Thus, durability of soil asphalt systems is an important variable which must be considered in an adequate evaluation procedure.

Several procedures are available which evaluate the effect of water on soil asphalt mixtures. Among these are alterations of the Hubbard-Field, Hveem, Marshall and unconfined compression tests which have been briefly discussed. The moisture-vapor susceptibility test and the immersion-compression tests will be discussed below as being representative of "standard" durability tests.

Moisture Vapor Susceptibility Test. Moisture vapor susceptibility test indicates the extent to which the treated soil will be affected by moisture vapor from wet sub-grades. Specimens prepared for test in the Hveem Stabilometer are subjected to a 75-hour moisture vapor treatment before stabilometer tests are conducted. A modified Resistance Value is calculated from stabilometer data. If there is a significant difference between the stability value after moisture vapor treatment as opposed to the stability value without treatment, the asphalt films will be replaced by water and a stability loss could be expected if used as a pavement layer subjected to water.

Immersion Compression Test. The immersion compression test measures the loss of cohesion resulting from the action of water on soil asphalt systems. Four-inch diameter by 4-inch specimens are prepared. One group of specimens is tested at 77°F in an unconfined compression test according to ASTM D 1074. "Method of Test for Compressure Strength of Bituminous Mixtures."

A summary of the suitability of the various test methods discussed above is shown in Table 2.

TEST CRITERIA FOR ACCEPTABLE PERFORMANCE

The majority of bituminous soil stabilization has been performed with asphalt cement, cutback asphalt and asphalt emulsion. Current design and construction trends, particularly in the state highway departments, have indicated that stabilization of base courses with asphalt cements is by far the most popular form of bituminous stabilization (26). In general, those materials which are most effectively stabilized with asphalt cement have lower percentages of fines than those materials which have been stabilized with cutback asphalt and emulsion.

Gradation Requirements. Some of the earliest criteria for bituminous stabilization were developed by the Highway Research Board Committee on Soil-Bituminous Roads. These criteria were revised and published by Winterkorn (33) and appear in Table 3. The American Road Builders Association (34) made similar recommendations and these are shown in Table 4.

The Asphalt Institute (35) grading and plasticity requirements for bituminous base course specifications require:

a. less than 25 percent passing the No. 200 sieve,

b. sand equivalent not less than 25, and

c. plasticity index less than 6.

Herrin has presented (36) and revised (37) a table (Table 5) recommending suitable soils for stabilization by bituminous materials. Contained in this table are recommendations on the suitability of various soils with certain percentages of minus No. 200 material, and certain liquid limit and plasticity index ranges.

Certain limits have been developed by the Asphalt Institute's Pacific Coast Division, Chevron Asphalt Company and Douglas Oil Company for emulsion treated materials. The requirements recommended by the Asphalt Institute (38) (Table 6) suggest that the percent of minus No. 200 material should be in a range of 3-15 percent, the plasticity index should be less than 6, and the product of the plasticity index and the percent passing the No. 200 sieve should not exceed 60. The Chevron Asphalt Company (39) has presented criteria (Table 7) which indicate that the California sand equivalent test should be used as a measure of the plasticity requirements for the soil and should have a minimum value of 30. Up to 25 percent passing the No. 200 sieve is allowed for the material identified as silty sand.

Dunning and Turner (40) of the Douglas Oil Company have presented guidelines for emulsion stabilization as shown in Table 8.

Materials Research and Development, Inc. of Oakland, California, has recently published a guide for asphalt stabilization for the U. S. Navy (41) in which criteria recommended by the Asphalt Institute and Chevron Asphalt Company have been utilized. This guide recommends that the maximum amount passing the No. 200 sieve should be less than 25 percent, the plasticity index less than 6, sand equivalent more than 30, and the product of the plasticity index and the percent passing the No. 200 sieve less than 72 in all cases. These criteria apply when both cutback asphalt and emulsified asphalt are used as soil stabilizers. The grading requirements (Table 9) for sands and semi-processed materials are identical to those recommended in Table 7 by Chevron Asphalt Company.

Grading requirements for materials to be stabilized with asphalt cement in a central plant have not been adequately defined. In general, those materials that are specified as suitable for asphalt concrete surface courses are more than adequate for base courses. Most asphalt treated base course specifications, however, will allow a larger maximum size of aggregate and the grading band is not as restrictive. A recent review of state highway specifications gives detailed information on these grading bands (27). For example, Texas (42) and California (43) have grading specifications as shown in Table 10. In addition, Texas specifies a maximum liquid limit of (41 and a maximum plasticity index of 16. The majority of the state highway departments recommended 12 percent or less passing the No. 200 sieve.

Air Force recommendations for gradings of materials suitable for asphalt cement treated base course are shown in Table 11 (44). Although gradations 6, 7, 8 and 9 are specifically recommended, it is believed that all gradations are practical, provided they are economically feasible.

Materials that are suitable for bituminous treatment include AASHO classified A-1-a, A-1-b, A-2-4, A-2-6, A-3, A-4 and low plasticity A-6 soils (45), and soils classified by the Unified Classification System as SW, SP, SW-SM, SP-SM, SW-SC, SP-SC, SM, SC, SM-SC, GW, GP, GW-GM, GP-GM, GW-GC, GP-GC, GM, GC and GM-GC provided certain plasticity and grading requirements are met.

In general if the plasticity index or the percent passing the No. 200 sieve exceeds the values cited above, then experience shows that the intimate mixing of the bitumen and soil necessary for satisfactory stabilization is nearly impossible.

STABILITY AND DURABILITY REQUIREMENT

As discussed above several laboratory test methods have been used to assist the engineer in determining the asphalt content of stabilized mixtures. For convenience these can be separated into:

1. Methods for use with hot-mix asphalt cement stabilized materials.
2. Methods for use with liquid asphalts (cutbacks and emulsions).

A recent Highway Research Board Committee Report (27) has summarized design methods and criteria used for coarse aggregate type hot plant mixed bases. As shown on Table 12 the Hveem and Marshall methods of design are in popular use, but the criteria vary from state to state. Several states indicated the use of Marshall stability and unconfined compressive strength; however, they did not indicate criteria. Three states (Oregon, Washington and Wyoming) indicated the use of modified immersion-compression tests.

Marshall method criteria utilized by the Air Force (46) are shown in Table 13. The criteria listed for asphaltic concrete binder course are suitable for use with coarse graded aggregate hot-mix base courses while the criteria for sand-asphalt should be used for these particular types of asphalt cement treated materials. The Air Force has indicated that the asphalt content determined by the Marshall method should be altered depending upon the Pavement Temperature Index and the Traffic Area (Table 14). However, these criteria were developed for surface courses and do not appear to be warranted for base courses.

The Asphalt Institute (47) recommends three popular criteria for use in hot-mix base course design (Table 15). Specifically, the Asphalt Institute recommends the same criteria that are utilized for surface courses, but the test temperature is 100°F rather than 140°F. This recommendation applies to regions having climatic conditions similar to those prevailing throughout most of the United States and provided the base is 4 inches or more below the surface. Existing information suggests that most base courses at this depth do not reach a temperature in excess of 100°F, and, therefore, the 100°F testing temperature has been selected. Additional data on pavement temperature will be presented later.

Zoepf (cited in reference 48) has also recommended Marshall criteria based on studies conducted in Germany (Table 16) while Lefebvre (49) presented similar Marshall criteria for liquid asphalt mixture (Table 17).

McDowell and Smith (50) have recently presented a design procedure based on unconfined compressive strength and air voids criteria for the selection of the asphalt content. Test methods Tex-126-E for black base is a result of this research. The black base methods include the effect of the rate of loading on the properties of asphalt treated materials.

Test criteria developed based on the Hveem stabilometer for emulsion mixtures is shown in Table 18. This table suggests criteria for both light and heavy traffic (38, 39, 51).

SELECTION OF TYPE OF BITUMEN

An indication of the type of bitumen to use for certain types of soils has been suggested by the Asphalt Institute (35), Herrin (36), the Navy (41), the Air Force (52) and Chevron Asphalt Company (39). The Asphalt Institute (35) suggestions are shown in Table 19 while the recommendations of Herrin (36), which are similar, are shown in Table 20.

The Navy's (41) method to select emulsions and cutback asphalts is shown in Table 21 and Figure 1, respectively. The selection of the particular type of emulsion is based on the percent of the soil passing the No. 200 sieve and the relative water content of the soil, while the selection of the particular type of cutback asphalt is based on the percent passing the No. 200 sieve and the ambient temperature of the soil. The basis of selection between these two general kinds of asphalt depends on which kind is more readily available for a particular job. Air Force (52) recommendations are very general in nature and indicate the MC-70, MC-250, MC-800, RC-70, RC-250, RC-800 cutbacks and SS-1 emulsions are normally used. Soils which possess some fines or natural binders and are well graded can be stabilized with medium curing cutbacks; however, the rapid curing cutbacks are preferred.

The selection of either a cationic or anionic emulsion should be based on the type of aggregate that is used. Mertens and Wright (54) have developed a method by which an aggregate can be classified (Figure 2) to indicate its probable surface charge and to determine the type of emulsion (anionic or cationic) that is more suitable for the particular type of aggregate (Figure 3). In general, Chevron recommends SS and MS type emulsions with damp or wet aggregate mixes.

SELECTION OF THE QUANTITY OF BITUMEN

Methods which have been used for the determination of asphalt content for stabilized materials can be conveniently separated into methods based on laboratory tests performed on the soil, methods based on laboratory tests performed on the soil-asphalt mixture and those based on a combination of these two. Those methods based on tests performed on the soil-asphalt mixtures have been adequately summarized above and only those methods based on aggregate gradation are discussed below.

The quantity of asphalt necessary to coat the surface of the soil particles can in general be expressed as follows:

$$A = SA \times t \times y_a$$

where:

A = percent asphalt

t = asphalt film thickness

SA = surface area of soil or aggregate

Y_a = unit weight of asphalt

This equation has been quantified empirically by the Asphalt Institute (35), Oklahoma Department of Highways (55), McKesson (56) and Bird (57).

The Oklahoma Equation (55) developed for cutback asphalts has the following form:

$p = k + 0.005\ (a) + 0.01\ (b) + 0.06\ (c)$

where:

p = percent of residual asphalt by weight of dry aggregate

a = percent mineral aggregate passing the No. 10 sieve

b = percent mineral aggregate passing the No. 40 sieve

c = percent mineral aggregate passing the No. 200 sieve

k = 1.5 if plasticity index $\leq$ 8 and 2.0 if plasticity index > 8.

The asphalt Institute (35) adopted a method for use with cutbacks and emulsions as follows:

1. Cutbacks

$p = 0.02\ (a) + 0.07\ (b) + 0.15\ (c) + 0.20\ (d)$

where:

p = percent of residual asphalt by weight of dry aggregate

a = percent of mineral aggregate retained on No. 50 sieve

b = percent of mineral aggregate passing No. 50 sieve and retained on No. 100 sieve

c = percent of mineral aggregate passing No. 100 sieve and retained on No. 200 sieve

d = percent pf mineral aggregate passing No. 200 sieve

2. Emulsions

$p = 0.05\ (a) + 0.1\ (b) + 0.5\ (c)$.

where:

p = percent by weight of asphalt emulsion, based on dry weight of mineral aggregate

a = percent of mineral aggregate retained on No. 8 sieve

b = percent of mineral aggregate passing No. 8 sieve and retained on the No. 200 sieve

c = percent of mineral aggregate passing the No. 200 sieve.

This equation has also been utilized by the Navy (41) for cutback stabilization.

McKesson's (56) formula, given below, is similar in form to the Asphalt Institute's formula:

$$P = 0.75\ (0.05A = 0.010B + 0.50C)$$

where:

P = percent of asphalt emulsion by weight of dry sand

A = sand retained on the No. 10 sieve in percent

B = sand passing the No. 10 sieve and retained on the No. 200 sieve in percent

C = sand passing the No. 200 sieve in percent

Bird (57) has presented two formulas to use depending on the percent passing the No. 200 sieve.

Formula (1) $T = 0.02F + 0.1C + r.$

(for use with sands having a minimum of 60 percent passing the No. 10 sieve and 5 to 12 percent passing the No. 200 sieve)

Formula (2) $T = 0.2F + 0.1D + 4$

(for use with sands having a miminum of 50 percent passing the No. 10 sieve and more than 12 percent passing the No. 200 sieve).

where:

T = pounds of emulsified asphalt per cubic foot of loose, dry aggregate

F = percent aggregate passing the No. 10 sieve

C = percent aggregate passing the No. 200 sieve

D = difference, plus or minus, between 24 and C above.

The California Centrifuge Kerosene Equivalent (CKE) Method is based on surface area as well as particle surface characteristics. The complete California CKE Method can be found in California Test Method 303 (58); however, a revised method has been suggested for use by the Navy (41). The CKE method is suitable for asphalt cement, cutback, and emulsified asphalt stabilized materials.

The Navy (41) has also suggested emulsion quantities to be used for certain soils based on the percent passing the No. 10 sieve and percent passing the No. 200 sieve (Table 22). The development of the table was based on surface area and void content theory.

TEST TEMPERATURE

Standard test temperature for most stability tests is 140°F. This test temperature is indicative of the maximum pavement surface temperature achieved in most climates. Although higher pavement temperatures have been recorded, these temperatures do not persist for a long period of time nor do they persist for many days of the year. Thus 140°F represents a reasonable maximum temperature.

Base courses and subbases which may be bituminous stabilized can be expected to have a lower maximum temperature because they are some distance from the surface of a pavement. The relative thicknesses of the base and surface courses vary, but generally the total thickness of the pavement is a function of vehicle loads and pavement design method among other factors. Surface course thickness requirements range from 2 inches to 6 inches. A reasonable average value for the thickness of asphalt concrete surface courses is 3 to 4 inches. Thus an examination of pavement temperatures below this depth would be useful in establishing an appropriate test temperature for bituminous stabilized base courses.

Pavement Temperature. Pavement temperatures were measured at the surface and at depths of 2, 4, 6, 8, 10 and 12 inches in a 12-inch section of asphalt concrete pavement at College Park, Maryland, from June 1, 1964 to May 31, 1965 by Kallas (59). This study indicated that the maximum temperature at the surface of the pavement was 142°F, while simultaneously at a 4-inch depth the temperature was 117°F (Figure 4). Maximum temperatures at greater depth were lower as expected. Analysis of the data by Kallas (59) indicates that the pavement surface temperature is above 140°F only a fraction of one percent of the time in the area under study. At a depth of four inches it was above 110°F only 1 percent of the time and above 100°F only 5 percent. However, during the months of June, July and August at a depth of four inches the pavement will remain above 100°F about 20 percent of the time. Kallas indicated that testing temperatures of above 110°F for 6-inch pavement depth and 100°F for 12-inch pavement depth may be appropriate. The authors consider these temperatures conservative even for temperate zones of the world.

A study conducted in Michigan during 1964 by Manz (60) (Figure 5) indicates that pavement temperatures 1/4 inch below the surface reached 130°F. At a depth of 5 1/4 inches the maximum pavement temperature was 108°F.

Straub et al. (61) measured temperatures on a pavement section at Potsdam, New York. These data indicate that the maximum temperatures at the surface and at depths of 1/4, 2, 4, 6, 8, 10 and 12 inches were 144, 131, 122, 111, 103, 98, 94, and 90°F, respectively. Additional analysis of the data by Straub (61) indicates that about 12 percent of the time during the month of July the temperatures at 4 inches in depth will be above 100°F. On a yearly basis the temperature at a 4-inch depth will be above 100°F about 3 percent of the time.

Rumney and Jimenez (62) and Long (63) have measured pavement temperature profiles in the southern United States (Figure 6). The data collected in Arizona (62) indicate that maximum surface temperatures and at 2, 4, 6, 8, 10 and 12 inch levels were 160, 142, 132, 123, 116, 113 and 111 respectively. During the hot months of July, August and September the surface temperature remained above 140°F from 7 to 22 percent of the time during a day. Based on these data the authors believe that modification in the stability test temperature may be beneficial: For example, laboratory stability evaluation of the top two inches might utilize a test temperature of 160°F in tropical or semi-tropical regions and for that part of the pavement between the 2-and 6-inch levels, the present 140°F testing temperature simulates the conditions experienced in the field. Below the 6-inch level a test temperature of 120°F should be considered for those regions of high insolation or solar flux.

Long (63) has presented temperature data for pavements containing two types of asphalt treated materials. These data indicate (Figure 7 and 8) that maximum temperatures of the order of 110 to 120°F can be expected at a depth of five inches. More recent data collected on a world wide basis are available (64, 65).

From a review of the published information it is noted that pavement temperatures at various pavement depths are functions of the regional climate, the weather and the specific location of the pavement among other factors. If a testing temperature is to be selected for base course mixture testing, a method for calculating the expected pavement temperature at various depths would be helpful. Methods which will allow the engineer to calculate pavement temperatures include those by Barber (66), Straub et al. (61) and Dempsey and Thompson (67). These methods involve the solution to a heat flow equation and typical inputs are as follows (66):

1. average air temperature,
2. daily range in temperature,
3. depth below surface,
4. solar radiation,
5. absorptivity of surface to solar radiation, and
6. material heat flow properties such as diffusivity, conductivity, specific heat.

<u>Selection of Test Temperature</u>. By selecting one of the above mentioned methods it would be possible to determine a fairly accurate maximum pavement temperature-depth relationship for a number of locations throughout the world. This information could then be utilized for selecting test temperatures for materials to be used at selected depths in a pavement.

In absence of this detailed development the curves of Figure 9 originated by Dormon and Metcalf (68) may be used or a base course test temperature of 100^{o}F should be considered for the cooler, northern climates, while 120^{o}F should be considered for the hotter, southern climates of the northern hemisphere.

LAYER EQUIVALENCY

The concept of layer equivalencies has been in use for a number of years by several agencies. The concept most often advanced is that of equating different types of roadbuilding materials in terms of equivalent thickness in a structural section. In the case of layer equivalencies for base courses, it is often the practice to express layer equivalencies in terms of equivalent thicknesses of granular base course. For example, the Asphalt Institute suggests that a 2 to 1 layer equivalency exists between granular base and hot mixed bituminous stabilized base. This statement implies that 1 inch of asphalt stabilized material will replace 2 inches of granular material assuming certain boundary conditions are satisfied.

The development of appropriate layer equivalencies has been a subject of a number of research projects. The general conclusion reached by these investigators is that a variety of methods exist to establish equivalencies for specific materials and specific pavement sections. These methods can also be used for general cases provided the investigator realizes that equivalencies generated will depend on:

1. Wheel load and contact pressure,
2. Stiffness characteristics of the particular material,
3. Stiffness characteristics of other materials in the structural section,

4. Subgrade characteristics,

5. Thickness of the various components of the structural sections, and

6. Position of the material in the structural section.

A brief review of selected literature pertaining to layer equivalencies will provide general information as to the magnitude of appropriate equivalencies for bituminous stabilized materials. The pavement thickness design equation developed from the AASHTO road test indicates that one inch of asphalt concrete is equivalent in performance to 3.1 inches of crushed rock base or 4.0 inches of gravel subbase. Following the development of the design equation, the AASHTO Design Committee, as part of their interim design procedure (69) suggested layer equivalencies for a range of asphalt-treated materials as shown in Table 23. From available information, it would appear that these values (other than those for asphalt concrete) are based on judgement rather than on the results of tests, since little or no performance data were available for a number of the materials listed (70).

Skook and Finn (71), in their analysis of the AASHTO Road Test data, indicated layer equivalencies of the asphalt concrete surfacing in terms of crushed-rock base ranged from slightly more than 2 to 6.7 depending on the criteria for evaluation. Typical results of their work is shown in Figure 10. For a conservative estimate these authors recommend a layer equivalency of asphalt concrete to crushed rock of 2.

Using compressive strain at the surface of the subgrade and radian strain on the underside of the asphalt-bound layer, Lettier and Metcalf (72) have established layer equivalencies for a series of subgrade conditions and thickness of untreated granular material and asphalt concrete. From these analyses (for an 18,000-lb. single axle load and 70-psi contact pressure), they demonstrated that the layer equivalency of asphalt concrete to untreated aggregate is dependent on the thickness of the asphalt layer and the stiffness characteristics of the subgrade. Typical values obtained from this study are shown in Figure 11. It should be emphasized that the results of Lettier and Metcalf as well as other researchers using similar methods of study are based on certain assumptions with respect to the properties of materials comprising the structural section and to axle load and contact pressure. It is not inconceivable that other values for equivalency could be obtained if other assumptions were used.

Terrel and Monismith (70) based on both laboratory and field test sections have established equivalencies for a variety of asphalt treated materials for both summer and winter conditions. These values are shown in Table 24 and are based on criteria similar to those utilized by Lettier and Metcalf.

The Chevron Asphalt Company research in the area of layer equivalencies for asphalt stabilized materials is aimed towards development of a rational pavement design method utilizing material properties obtained from repeated load tests, layered elastic computer programs, and appropriate failure criteria. Equivalencies, as a function of traffic (DTN) and resilient modulus are shown in Figure 12 (73).

The development of layer equivalencies has been studied by a number of the investigators, some of which are given in Reference 74 to 79. In general the values are in the range indicated above. Examples of the use of this equivalency in design procedures are discussed below.

The State of California modified its pavement design procedure somewhat on the basis of the AASHTO road test results and increased the "gravel-equivalency" factor for asphalt concrete over that used for the period 1950 to 1963. This gravel-equivalency factor (layer equivalency) is expressed in terms of subbase-type material (gravel rather than crushed rock) and, in the case of asphalt concrete is expressed in terms of traffic intensity and thickness of layer. In the design procedure developed by the State of California, the equivalency factor varies from 2.5 for residential traffic to 1.6 for heavy industrial traffic (Table 25) (80).

Layer equivalencies utilized in the Asphalt Institute design method (81) are summarized in Table 26, while Table 27 illustrates the equivalencies utilized by the State of Oklahoma (82). Layer equivalencies for a number of other states, expressed in the structural layer coefficients compatible with the AASHTO design methods, are shown in Table 28 (83). In general it should be noted that these equivalencies are conservative relative to those developed by theoretical analyses of test roads and pavement sections.

The Texas Method of pavement design utilizing the triaxial test as described in reference 84 considers stabilized layers by correcting for tensile strength of the improved base course by use of the cohesiometer test. The triaxial method as described in reference 85, however, doesn't make this correction and, thus, several thick sections of pavements containing asphalt stabilized base courses have been constructed in Texas.

A new pavement design being implemented in Texas (86), however, has the ability to consider the supporting capacity of bituminous stabilized materials. The performance equation utilized in this system has been used to develop layer equivalencies as described below.

Selected bituminous stabilized pavement sections were evaluated with the dynaflect to determine their stiffness coefficients. These coefficients are summarized in Table 29 and can be used in the following performance equation (86)

$$P = 5 - \left[5 - P_1 + 53.6 \frac{NS^2}{\alpha} \right]^2$$

where:

P = final serviceability index

P_1= initials

N = number of 18-kip single axle loads applied

S = surface curvature index determined by the Dynaflect and dependent upon layer stiffness coefficients

α = temperature factor

together with the deflection equation to determine layer equivalencies.

These equations were utilized in the flexible pavement design method with the following inputs:

1. initial serviceability index = 4.2;
2. final serviceability index = 3.0;
3. surface thickness = 1.5 inches, surface stiffness coefficient = 1.0;
4. base thickness variable, base stiffness coefficient variable;
5. subbase thickness = 6.0 inches, subbase stiffness coefficient = 0.40;
6. subgrade stiffness coefficient variable; and
7. temperature constant variable.

The calculated values indicate an equivalency between 2.0 and 3.0 for most of the courses investigated. As the average temperature increases, the equivalency factor decreases as can be predicted from layered elastic solutions with appropriate material constants. However, as the traffic increases and the subgrade strength increases, the layer equivalency increases which is contrary to the literature cited above (72, 73, 80).

The above literature review indicates layer equivalencies for a variety of materials including a variety of forms of asphalt stabilized materials. The relative equivalencies of some commonly utilized asphalt stabilized materials are presented therein. The importance of proper construction of asphalt stabilized materials with liquid asphalts both plant mix and road mix is emphasized. The actual magnitude of these equivalencies must be more accurately defined by additional field testing if valid comparisions are to be made.

ECONOMIC COMPARISIONS

A valid economic comparision of alternate base course materials must be made on both initial cost and maintenance cost. Since little reliable maintenance cost information is presently available, this report will compare the economics of base courses on initial cost only.

The cost of asphalt stabilized base courses like the cost of all road building materials has escalated during the last 18 months. The monthly low bids for black base as received by the Texas Highway Department (88) for the period May 1973 to May 1974 are shown on Figure 13. A similar trend has existed for asphalt concrete (Figure 14) while Figure 15 compares the increase cost trend of both asphalt concrete and black base. The flexible base whose cost information is shown on Figure 15 is of moderate quality and was added to the figure to illustrate a price trend rather than for cost comparison.

Average low bid prices for May 1974 for a variety of road building materials are shown in Table 31 (88) while typical price ranges for base course materials for various projects around the state taken from bid summary sheets are shown in Table 32 (89).

From a review of the above information, it appears as if the present price of asphalt concrete will be 17 to 19 dollars per ton, black base 15 to 17 dollars per ton and good quality flexible base 5 to 7 dollars per ton. (All of the prices are for materials in place.) Thus, it appears as if black base is and has remained about 2 dollars per ton less expensive than asphalt concrete and good quality flexible base about 10 dollars per ton less expensive than black base. It should also be noted that the cost of asphalt concrete and black base has increased at a more rapid rate than the untreated or so-called flexible base, although this is not clearly indicated in Figure 15. A review of cost information in 1972 further indicates the apparent trend as the average price of asphalt concrete was in the range of 6 to 8 per ton, black base 5 to 7 dollars per ton and good quality flexible bases 3.50 to 5.50 per ton.

Why has the cost of the bituminous treated materials escalated at a much more rapid rate than the untreated flexible base courses? The cost of asphalt has increased from about 30 dollars per ton to nearly 100 per ton. Fuel costs to heat and dry aggregate, heat asphalt and transport materials have increased.

A review of the component cost of asphalt concrete (Table 33) indicates that material cost accounts for about 50 percent (87) of the total cost of asphalt concrete. For a mixture containing 6 percent asphalt, the cost of asphalt cement would be $1.80 per ton of hot mix with asphalt cement priced at $30.00 per ton and $6.00 per ton of hot mix with asphalt cement priced at $100.00 per ton. Further assuming that aggregate cost were about $1.80 per ton when asphalt was priced at $30.00 per ton (1972 cost figures) and that the price of aggregates has escalated about 50 percent (Figure 15), it can be shown that materials costs can account for about $5.00 of the cost increase of hot mix. Thus, $5.00 of the $12 to $14 increase in the cost of hot mix can be attributed to materials with $7 to 9 per ton to be attributed to such factors as plant expenses, transportation, laydown and profit.

From a review of the component cost of hot mix (Table 33) it appears as if significant savings can be effected by reducing material cost as some 50 percent of this total cost of hot mix can normally be attributed to the production of hot mix. However, from the preceding paragraph, it is apparent that other factors have become increasingly more important and should be investigated.

DISCUSSION OF RESULTS

As discussed previously, a number of mixture characteristics must be considered to properly evaluate bituminous treated mixtures including stability, durability, fatigue behavior, tensile behavior, flexibility and workability. Ideally a single test would provide sufficient information, however, such a test has not been developed nor is there hope for such a test in the near future. Thus, it appears as if a number of tests must be considered to adequately define mixture characteristics.

Test geometry and loading conditions of the ideal test must be such that they nearly represent the state of loading encountered in the field by the mixture. Certainly the state of stress in the field is biaxial if not triaxial while the load is repeated and of varying magnitude and duration. Research has indicated that a testing apparatus to perform such a test and the theory necessary to interpret such test results are complex and in the near future will not be practical for everyday use. Thus, less complex tests must be considered and their results correlated with in-service performance of pavements.

Basically the engineer would prefer a test to be suitable for construction control and mixtures evaluation as well as for utilization in pavement design procedures to determine layer thickness. Thus, it is important that the procedure have the capability to delineate between an acceptable and unacceptable mixture for all of these purposes.

Initial work in the follow-on study resulting from this Type B study will investigate alternative testing techniques in order to best define the requirements of a test method as described above. The review of the test method presently being utilized and included in this paper will be used as background data with some type of repeated load test appearing to be most desirable.

Those materials most suitable for bituminous stabilization have been defined. The gradations and Atterberg Limits suggested by Herrin (36) (Table 5) appear to be reasonable. The utilization of the sand equivalent test together with Atterberg Limits and sieve analyses should be used as the preliminary criteria for soil stabilization followed by laboratory testing. Criteria for acceptance of mixture based on laboratory testing need to be further defined for bituminous stabilized materials. Testing temperatures as well as acceptance criteria should be estiblished for existing tests as well as any developed tests based on field performance.

The concept of layer equivalency ideally should be applied to industrial projects as the layer equivalency is dependent on wheel load and contact pressure, stiffness characteristics of the particular material, stiffness characteristics of other materials in the structural section, subgrade characteristics, thickness of the barious components of the structural sections and position of the material in the structural section. Typical equivalencies of black base as determined from the literature review are 2:1.

A review of the component cost of hot mix has suggested that materials costs have been a rather large portion of the costs of bituminous treated materials, thus, investigating cheaper materials is an attractive area of study. The price of asphalt has doubled during the last 12 months and thus has assumed a somewhat larger proportion of the component cost of hot mixed bituminous materials. Cost savings thus may be effected by reducing the amount of asphalt.

Aggregate costs have escalated about 50 percent in the last 12 months. Alternate sources of aggregates such as sands appear to be promising in many areas of Texas as substitutes for the conventional black base aggregates. Other "marginal materials" (As defined by present specifications criteria) should be investigated for potential utilization.

Dryer drum mixing operations are becoming more popular for jobs requiring large tonnages of hot mixed bituminous materials.

The potential cost saving by use of this type of equipment should be between fifty cents to one dollar. Other types of mixing, transport and laydown equipment should be investigated with the hope of reducing these non-material costs.

FUTURE RESEARCH PLAN

The Type A study 2-8-74-41 titled Bituminous Treated Bases which was a follow-on project to this study will be seeking to find ways of reducing the cost of black base. Aggregates have been obtained from Districts 5, 11, 13, 16, 20, 21 and 25 from district laboratory personnel for the study. These materials are in relatively large supply and can be obtained at reasonable cost. Gradation and Atterberg Limits have been obtained and are shown in Table 34. The properties of these materials blended with various percentages of asphalt cement will be determined. These properties which will include strength and durability properties will be utilized to design typical pavement sections from which cost comparisons can be made. The object of the inclusion of the pavement design portion of the study will be to define the conditions under which certain types of materials will be economically competitive while providing the same predicted performance.

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Table 1. Types of Bituminous Bound Base Courses Utilized by State Highway Departments in the United States

Type Bituminous Treated Base	Percent of Total Production
Coarse Aggregate Hot Plant Mix	70
Fine Aggregate Hot and Cold Plant Mix	9
Coarse Aggregate Cold Plant Mix	6
Mixed in Place	3
Penetration Macadam	1

after reference 26

TABLE 2. SUMMARY OF TEST METHODS

Test Method	Properties Measured	Complexity of Test Method	Suitability of Test Geometry and Specimen Preparation Method	Suitable For Use In: Field Control	Suitable For Use In: Mixture Evaluation	Suitable For Use In: Pavement Design
Sieve Analysis	Gradation Workability	Simple	---	Yes	?	No
Hubbard-Field	Stability Durability	Simple	Poor	Yes	Yes	No
Hveem	Stability Durability	Simple	Good	Yes	Yes	No
Marshall	Stability Durability Tensile	Simple	Good	Yes	Yes	No
Unconfined Compression	Stability Durability	Simple	Fair	Yes	Yes	No
Triaxial	Stability	Complex	Good	?	Yes	Yes
Repeated Load Triaxial	Stability Flexibility	Complex	Very Good	No	Yes	Yes
CBR	?	Simple	Poor	Yes	?	No
Iowa Bearing Value	?	Simple	Poor	Yes	?	No
Florida Bearing Value	?	Simple	Poor	Yes	?	No

TABLE 3

TYPES OF SOIL BITUMEN AND CHARACTERISTICS OF SOILS

EMPIRICALLY FOUND SUITABLE FOR THEIR MANUFACTURE

Sieve Analysis	Soil Bitumen, † %	Sand Bitumen, %	Waterproofed Granular Stabilization, %		
Passing:			A	B	C
1 1/2-in.	...	...	100		
1-in.	‡	...	80–100	100	
3/4-in.	...	...	65–85	80–100	100
No. 4	>50	100	40–65	50–75	80–100
No. 10	...	...	25–50	40–60	60–80
No. 40	35–100	...	15–30	20–35	30–50
No. 100	...	...	10–20	13–23	20–35
No. 200	10–50	< 12; < 25 § ‖	8–12	10–16	13–30
Characteristics of Fraction Passing No. 40 Sieve					
Liquid limit	< 40	...	...	...	...
Plasticity index	< 18	...	< 10; < 15	< 10; < 15	< 10; < 15 ¶
Field moisture equiv.	...	< 20 §	...	...	...
Linear shrinkage	...	< 5 §	...	...	...

† Proper or general.

‡ Maximum size not larger than 1/3 of layer thickness; if compacted in several layers, not larger than thickness of one layer.

§ Lower values for wide and higher values for narrow gradation band of sand. If more than 12% passes, restrictions are placed as indicated on field moisture equivalent and linear shrinkage.

‖ A certain percentage of -200 or filler material is indirectly required to pass supplementary stability test.

¶ Values between 10 and 15 permitted in certain cases.

[after Winterkorn (33)]

TABLE 4

GRADING AND PLASTICITY REQUIREMENTS

FOR SOIL-BITUMEN MIXTURES

Sieve Size	Percent Passing
No. 40	50 - 100
No. 200	0 - 35
Atterberg Limits	**Maximum Value**
Liquid limit	30
Plasticity index	10

[after American Road Builders Association (34)]

TABLE 5

ENGINEERING PROPERTIES OF MATERIALS

SUITABLE FOR BITUMINOUS STABILIZATION

% Passing Sieve	Sand-Bitumen	Soil-Bitumen	Sand-Gravel-Bitumen
1-1/2"			100
1"	100		
3/4"			60-100
No. 4	50-100	50-100	35-100
10	40-100		
40		35-100	13-50
100			8-35
200	5-12	good - 3-20 fair - 0-3 and 20-30 poor - > 30	0-12
Liquid Limit		good - < 20 fair - 20-30 poor - 30-40 unusable - > 40	
Plasticity Index	< 10	good - 5 fair - 5-9 poor - 9-15 unusable - > 12-15	< 10

Includes slight modifications later made by Herrin.

[after Herrin (36)]

TABLE 6

GRADING, PLASTICITY AND ABRASION REQUIREMENTS FOR

SOILS SUITABLE FOR EMULSIFIED ASPHALT TREATED BASE COURSE

Sieve Size	Percent Passing by Weight		
	2 inch maximum	1-1/4 inch maximum	3/4 inch maximum
2-1/2 inch	100		
2 inch	90-100	100	
1-1/2 inch		90-100	
1 inch			100
3/4 inch	50-80	50-80	80-100
No. 4	25-50	25-50	25-50
No. 200	3-15	3-15	3-15

Other Requirements

a. Plasticity Index 6 maximum
b. Resistance Value 75 minimum
c. Loss in Los Angeles Abrasion Machine 50 percent maximum
d. Product of Plasticity Index and the percent passing the No. 200 sieve shall not exceed 60.

[after The Asphalt Institute, Pacific Division (38)]

TABLE 7

TYPICAL AGGREGATES SUITABLE FOR TREATMENT WITH EMULSIFIED ASPHALTS

Category	ASTM Test Method	Processed* Dense Graded Aggregates	SANDS			Semi-Processed Crusher, Pit or Bank Run Aggregates
			Poorly	Well Graded	Silty Sands	
Gradation: 1 1/2"		100				100
% Passing 1"		90-100				80-100
3/4"		65-90				—
1/2"		—	100	100	100	—
No. 4		30-60	75-100	75-100	75-100	25-85
16	C-136	15-30	—	35-75	—	—
50		7-25	—	15-30	—	—
100		5-18	—	—	15-65	—
200		4-12	0-12	5-12	12-25	3-15
Sand Equivalent, %	D-2419	30 Min.	30 Min.	30 Min.	30 Min.	30 Min.
Plasticity Index	D-424	—	NP	NP	—	—
Untreated Resistance R Value	**	78 Min.	60 Min.	60 Min.	60 Min.	60 Min.
Loss in Los Angeles Rattler (after 500 revolutions)	C-131	50 Max.	—	—	—	60 Max.

*Must have at least 25% Crush Count
**See AASHO T-174, T-175, and T-176

[after Chevron Asphalt Co. (39)]

TABLE 8

GUIDELINES FOR EMULSIFIED ASPHALT STABILIZATION

Test	Requirements		
	Good	Fair	Poor
% passing No. 200 sieve	3 - 20	0 - 3, 20 - 30	>30
Sand Equivalent	>25	15 - 25	<15
Plasticity Index	< 5	5 - 7	> 7

[after Dunning and Turner (39)]

TABLE 9

GRADING REQUIREMENTS FOR SANDY AND SEMI-PROCESSED MATERIALS

Sieve Size	Percent passing sieve for soils that are:			
	Poorly-graded sands	Well-graded sands	Silty sands	Semi-processed*
1 1/2"	---	---	---	100
1"	---	---	---	80 - 100
3/4"	---	---	---	---
1/2"	100	100	100	---
No. 4	75 - 100	75 - 100	75 - 100	25 - 85
No. 16	---	35 - 75	---	---
No. 50	---	15 - 30	---	---
No. 100	---	---	15 - 65	---
No. 200	0 - 25	5 - 12	12 - 25	3 - 15

*Semi-processed crusher, pit, or bank-run aggregates.

[after U. S. Navy (41)]

Table 10. Typical Asphalt Cement Treated Base Course Requirement

Sieve Size	Percent Passing by Weight			
		Texas		
	California	Grade 1	Grade 2	Grade 3
1 3/4 inch			100	100
1 1/2 inch		100	90-100	
1 1/4 inch	100			
1 inch	95-100	90-100		
3/4 inch	80-95			
3/8 inch	50-65	45-70		
No. 4	35-50	30-55	25-55	
No. 30	12-25			
No. 40		15-30	15-40	15-40
No. 200	2-7			

[after references (42) and (43)]

TABLE 11

AGGREGATE GRADATION SPECIFICATION LIMITS FOR BITUMINOUS PAVEMENTS

Percentage by Weight (Passing)

Surface Course

Sieve Designation (Square Openings)	1-1/2-in. Maximum			1-in. Maximum			3/4-in. Maximum			1/2-in. Maximum			3/8-in. Maximum		
	Gradation 1			Gradation 2			Gradation 3			Gradation 4			Gradation 5		
	a	b	c	a	b	c	a	b	c	a	b	c	a	b	c
1-1/2-in.	100	100	100	---	---	---									
1-in.	79-95	83-96	86-98	100	100	100	---	---	---						
3/4-in.	---	---	---	80-95	84-96	90-98	100	100	100	---	---	---			
1/2-in.	61-75	66-79	71-84	68-86	74-89	79-93	80-95	84-96	87-98	100	100	100	---	---	---
3/8-in.	---	---	---	---	---	---	---	---	---	79-94	81-95	86-96	100	100	100
No. 4	42-54	48-60	54-66	45-60	52-68	60-75	55-70	61-74	67-80	59-73	64-86	72-83	75-95	78-95	80-95
No. 10	31-43	37-49	43-55	32-47	39-54	47-62	40-54	46-60	54-66	43-57	50-64	57-70	56-76	60-80	62.84
No. 40	16-25	20-29	25-34	16-26	21-32	26-37	22-31	26-35	31-40	23-33	27-37	31-42	26-44	29-47	32-50
No. 80	10-17	12-19	15-22	10-18	13-21	15-24	12-20	15-23	19-26	13-20	16-23	19-26	14.28	16-30	18-32
No. 200*	3-6	3.5-6.5	4-7	3-7	3.5-7.5	4-8	3-7	3.5-7.5	4-8	4-8	4-8	4-8	5-9	6-10	7-11

Binder Course

Sieve Designation (Square Openings)	Gradation 6			Gradation 7			Gradation 8			Gradation 9		
	a	b	c	a	b	c	a	b	c	a	b	c
1-1/2-in.	100	100	100	---	---	---						
1-in.	73-95	75-95	79-95	100	100	100	---	---	---			
3/4-in.	---	---	---	72-95	75-95	81-96	100	100	100	---	---	---
1/2-in.	55-73	59-77	62-80	61-82	65-85	69-89	70-95	74-95	77-95	100	100	100
3/8-in.	---	---	---	---	---	---	60-80	64-84	68-88	71-95	75-95	78-95
No. 4	35-51	39-55	42-58	38-54	43-59	48-66	42-60	47-65	52-70	50-71	54-75	59-80
No. 10	23-38	27-42	31-46	25-41	29-45	34-50	28-46	33-51	36-54	32-53	36-57	41-62
No. 40	11-21	13-23	15-25	12-23	14-25	17-28	14-26	16-28	18-30	16-29	18-31	21-34
No. 80	6-14	7-15	8-16	7-16	8-17	10-18	8-18	9-19	10-20	10-20	11-21	12-22
No. 200*	3-7	3-7	3-7	3-7	3-7	3-7	3-7	3-7	3-7	4-9	4-9	4-9

All High-pressure Tire and Tar-rubber Surface Courses

Sieve Designation (Square Openings)	Gradation 10			Gradation 11		
	a	b	c	a	b	c
1-in.	100	---	---	---	---	---
3/4-in.	84-97	---	---	100	---	---
1/2-in.	74-88	---	---	82-96	---	---
3/8-in.	68-82	---	---	75-90	---	---
No. 4	54-67	---	---	60-73	---	---
No. 10	38-51	---	---	43-57	---	---
No. 20	26-39	---	---	29-43	---	---
No. 40	17-30	---	---	19-33	---	---
No. 80	9-19	---	---	10-20	---	---
No. 200*	3-6	---	---	3-6	---	---

[after U. S. Army (44)]

TABLE 12

DESIGN METHODS AND CRITERIA FOR COARSE AGGREGATE HOT MIX BASE COURSES

A. Hveem Method

State	Stability	Percent Air Voids	Percent Voids Filled With Asphalt	Cohesiometer
California	35 minimum	4-6		
Colorado	30-45	3-5	80-85	
Hawaii	35 minimum	5-10	75	300 minimum
Nevada	30-37 min.	3-5		
Oklahoma	35 minimum	8 maximum		
Oregon	30 minimum	10 maximum		150 minimum
Texas	30 minimum			
Washington	20 minimum			50 minimum

B. Marshall Method

State	Stability lbs.	Flow Value 0.01 in.	Percent Air Voids	Percent Voids Filled With Asphalt
District of Columbia	750 minimum	8-16	3-8	65-75
Georgia	1800 minimum	8-16	3-6	65-75
Kansas	800-3000	5-15	1-5	70-85
Kentucky	1100-1500	12-15	4-6	
Mississippi	1600	16 maximum	5-7	50-70
New Jersey	1100-1500	6-18	3-7	
N. Carolina	800	7-14	3-8	
N. Dakota	400 minimum	8-18	3-5	
Pennsylvania	700 minimum	6-16		60-85
Rhode Island	750 minimum		3-8	
S. Carolina	1200-3000	6-12		
S. Dakota		8-18	3-5	
Wyoming	100 minimum			

C. Unconfined Compressive Strength

State	Load, psi	Percent Air Voids	Percent Voids Filled With Asphalt
Colorado	200-400	3-5	80-85
Oregon	150 minimum		

[after Highway Research Board (26)]

TABLE 13

CRITERIA FOR DETERMINATION OF OPTIMUM BITUMEN CONTENT
(Marshall Method)

Test Property	Type of Mix	Point on Curve: For 100 psi tires[1]	Point on Curve: For 200 psi tires[1]	Criteria: For 100 psi tires[1]	Criteria: For 200 psi tires[1]
Stability	Asphaltic-concrete surface course	Peak of curve	Peak of curve	500 lb or higher	1800 lb or higher
	Asphaltic-concrete binder course	Peak of curve[2]	Peak of curve[2]	500 lb or higher	1800 lb or higher
	Sand asphalt	Peak of curve		500 lb or higher	
Unit weight	Asphaltic-concrete surface course	Peak of curve	Peak of curve	Not used	Not used
	Asphaltic-concrete binder course	Not used	Not used	Not used	Not used
	Sand asphalt	Peak of curve		Not used	Not used
Flow	Asphaltic-concrete surface course	Not used	Not used	20 or less	16 or less
	Asphaltic-concrete binder course	Not used	Not used	20 or less	16 or less
	Sand asphalt	Not used	Not used	20 or less	16 or less
Percent voids total mix	Asphaltic-concrete surface course	4 (3)	4 (3)	3-5 (2-4)	3-5 (2-4)
	Asphaltic-concrete binder course	5 (4)	6 (5)	4-6 (3-5)	5-7 (4-6)
	Sand asphalt	6 (5)	-- (-)	5-7 (4-6)	--- (---)
Percent voids filled with bitumen	Asphaltic-concrete surface course	80 (85)	75 (80)	75-85 (80-90)	70-80 (75-85)
	Asphaltic-concrete binder course	70 (75)	60 (65)[2]	65-75 (70-80)	70-80 (55-75)
	Sand asphalt	70 (75)	-- (--)	65-75 (70-80)	-- (--)

[1] Figures in parentheses are for use with bulk impregnated specific gravity (water absorption greater than 2.5 percent).

[2] If the inclusion of asphalt contents of these points in the average causes the voids to fall outside the limits, then the optimum asphalt content should be adjusted so that the voids total mix are within the limits.

[after U. S. Air Force (46)]

TABLE 14

BITUMEN CONTENT AND PENETRATION GRADE OF ASPHALT FOR VARIOUS TEMPERATURE INDEX RANGES

		Bitumen Content by Traffic Areas								
		Type A Traffic Areas			Types B and C Traffic Areas			Type D Traffic Areas (2)		
Pavement Temp. Index	Asphalt Pen. Grade	Light Load Pavements	Intermediate Load Pavements (1)	Heavy Load Pavements	Light Load Pavements	Intermediate Load Pavements	Heavy Load Pavements	Light Load Pavements	Intermediate Load Pavements	Heavy Load Pavements
Negative	120-150	---	Optimum	(3)	Opt. +10%	Opt. +10%	Optimum	---	Opt. +10%	Opt. +10%
0-40	100-120	---	Optimum	(3)	Optimum	Optimum	Opt. -10%	---	Opt. +10%	Opt. +10%
40-100	85-100	---	Opt. -10%	(3)	Optimum	Optimum	Opt. -20%	---	Opt. +10%	Optimum
Above 100	60-70	---	Opt. -20%	(3)	Optimum	Opt. -10%	(3)	---	Optimum	Optimum

(1) Intermediate load pavements, for the purposes of this tabulation, include those for the twin bicycle, twin tricycle, and twin-tandem tricycle gear configurations for which design criteria are included in this manual.
(2) Blast zones within overrun areas are included with type D traffic areas.
(3) Design bitumen content to be furnished by OCE at time of airfield design.

PAVEMENT TEMPERATURE INDEX:

The sum, for a one-year period, of the increments above 75°F of monthly averages of the daily maximum temperatures. Average daily maximum temperatures for the period of record should be used where 10 or more years of record are available. For records of less than 10-year duration the record for the hottest year should be used. A negative index results when no monthly average exceeds 75°F. Negative indices are evaluated merely by subtracting the largest monthly average from 75°F.

[after U. S. Air Force (46)]

TABLE 15

MIXTURE DESIGN CRITERIA

A. Marshall Design Criteria

Traffic Category	Heavy		Medium		Light	
Test Property	Min.	Max.	Min.	Max.	Min.	Max.
No. of Compaction Blows Each End of Specimen	75		50		35	
Stability, all mixtures	750	---	500	---	500	---
Flow, all mixtures	8	16	8	18	8	20
Percent Air Voids						
Surfacing or Leveling	3	5	3	5	3	5
Base	3	8	3	8	3	8
Percent Voids in Mineral Aggregate						

B. Hveem Design Criteria

Traffic Category	Heavy		Medium		Light	
Test Property	Min.	Max.	Min.	Max.	Min.	Max.
Stabilometer Value	37	---	35	---	30	---
Cohesiometer Value	50	---	50	---	50	---
Swell	less than 0.030 inch					

C. Hubbard-Field Design Criteria

Traffic Category	Heavy		Medium and Light	
Test Property	Min.	Max.	Min.	Max.
Stability-Pounds	2,000	---	1,200	2,000
Percent Air Voids	2%	5%	2%	5%

Hot-mix asphalt bases, which do not meet the above criteria when tested at 140°F., should be satisfactory if they meet the criteria when tested at 100°F. and are placed 4 inches or more below the surface. This recommendation applies only to regions having climatic conditions similar to those prevailing throughout most of the United States. Guidelines for applying for the lower test temperature in regions having more extreme climatic conditions are being studied.

[after The Asphalt Institute (47)]

TABLE 16

MARSHALL MIX DESIGN CRITERIA

FOR ASPHALT CEMENT TREATED BASE COURSE

Marshall Requirement at 140°F	Traffic, Vehicles per day			
	Light (less than 3000)	Medium (1000-3000)	Heavy (3000-6000)	Extra Heavy (greater than 6000)
Stability, min.	330	440	550	660
Flow (0.01 in.)	4-20	4-18	4-16	4-14
Percent air voids	2-15	2-15	3-12	3-10

[after Zoepf as cited in (48)]

TABLE 17

MARSHALL MIX DESIGN CRITERIA FOR

CUTBACK AND EMULSIFIED ASPHALT MIXTURES

Marshall Test	Criteria for a Test Temperature of 77°F	
	Minimum	Maximum
Stability, lbs.	750	---
Flow, (0.01 in.)	7	16
Percent air voids	3	5

[after Lefebvre (49)]

TABLE 18

HVEEM MIX DESIGN CRITERIA

EMULSIFIED ASPHALT MIXTURES

	Criteria		
	Resistance Value		Moisture Pickup
	Before MVS*	After MVS*	During MVS, per cent
Asphalt Institute (38)	70 min.	60 min.	---
Chevron Asphalt Company (39)	---	70**, 78***	5.0 max.
Finn, et al. (51)	---	70**, 73***	5.0 max.

*Moisture Vapor Susceptibility
**Light Traffic
***Heavy Traffic

TABLE 19

SUITABLE TYPES OF BITUMEN FOR STABILIZATION

Type of Soils	Cutback Asphalts	Emulsions
Open-graded aggregate	RC-250, RC-800	MS-2
Well-graded aggregate with little or no fine aggregate and material passing the No. 200 sieve	RC-250, RC-800 MC-250, MC-800 SC-250, SC-800	MS-2 CMS-2 SS-1, CSS-1
Aggregate containing a considerable percentage of fine aggregate and material passing the No. 200 sieve	MC-250, MC-800 SC-250, SC-800	SS-1, SS-1h CSS-1, CSS-1h MS-2 CMS-2

[after the Asphalt Institute (35)]

*Asphalt Materials are specified according to ASTM Specifications (29)

Table 20. Suitable Types of Bituminous Materials*

Sand-Bitumen	Soil-Bitumen	Crushed Stone and Sand-Gravel-Bitumen
Hot Mix:	Cold Mix:	Hot Mix:
AC-5, AC-10	RC-2, RC-250, RC-3 MC-70, 250, 800	AC-5, AC-10
Cold Mix:		Cold Mix:
RC-2, RC-250, RC-3 MC-250, MC-800		RC-2, RC-250, RC-3 MC-250, 800
Emulsions:	Emulsions:	Emulsions:
EA-11M, EA-10S	EA-11M, EA-10S	EA-11M, EA-10S
EA-CSS-1, EA-CSS-1h	EA-CSS-1, EA-CSS-1h	EA-HVMS, EA-HVMS-90
		EA-CMS-2, EA-CMS-2h
		EA-CSS-1, EA-CSS-1h

*Asphalt materials are specified according to Texas Highway Department Specification (53)

[adopted after Herrin (36)]

Table 21. Selection of Type of Emulsified Asphalt for Stabilization**

Percent Passing No. 200 Sieve	Relative Water Content of Soil	
	Wet (5%+)	Dry (0-5%)
0-5	SS-1h (or CSS-1h)	CMS-2 (or SS-1h*)
5-15	SS-1, SS-1h (or CSS-1, CSS-1h)	CMS-2 (or SS-1h*, SS-1*)
15-25	SS-1 (or CSS-1)	CMS-2

**Asphalt materials are specified according to ASTM specifications.

*Soil should be pre-wetted with water before using these types of emulsified asphalts.

[after U.S. Navy (41)]

TABLE 22

EMULSIFIED ASPHALT REQUIREMENT

Percent passing No. 200	Lbs. of an emulsified asphalt per 100 lbs. of dry aggregate when percent passing No. 10 sieve is:					
	50*	60	70	80	90	100
0	6.0	6.3	6.5	6.7	7.0	7.2
2	6.3	6.5	6.7	7.0	7.2	7.5
4	6.5	6.7	7.0	7.2	7.5	7.7
6	6.7	7.0	7.2	7.5	7.7	7.9
8	7.0	7.2	7.5	7.7	7.9	8.2
10	7.2	7.5	7.7	7.9	8.2	8.4
12	7.5	7.7	7.9	8.2	8.4	8.6
14	7.2	7.5	7.7	7.9	8.2	8.4
16	7.0	7.2	7.5	7.7	7.9	8.2
18	6.7	7.0	7.2	7.5	7.7	7.9
20	6.5	6.7	7.0	7.2	7.5	7.7
22	6.3	6.5	6.7	7.0	7.2	7.5
24	6.0	6.3	6.5	6.7	7.0	7.2
25	6.2	6.4	6.6	6.9	7.1	7.3

*50 or less.

[after U. S. Navy (41)]

TABLE 23

LAYER EQUIVALENCIES OF ASPHALT-TREATED MATERIALS

AASHO INTERIM GUIDE*

Material	Layer Equivalency in Terms of:	
	Gravel	Crushed Rock
Surface Course:		
Road Mix (low stability)	1.8	1.4
Plant Mix (high stability)	4.0	3.1
Sand Asphalt	3.6	2.8
Base Course:		
Bituminous-Treated (coarse-graded)	2.7	2.1
Bituminous-Treated (sand asphalt)	2.3	1.8

*Data was adapted from information presented in AASHO Interim Guide.

TABLE 24

LAYER EQUIVALENCY VALUES BASED ON LIMITING STRESS

IN THE SUBGRADE AND STRAIN

IN THE ASPHALT CONCRETE

Base Material	Summer	Winter
Asphalt concrete	1.00	1.00
Untreated aggregate	2.20	25.00*
SM-K treated aggregate (uncured)	2.00	14.00
SM-K treated aggregate (cured)	1.20	1.50
MC-800 treated aggregate (uncured)	1.80	2.00
MC-800 treated aggregate (cured)	1.55	2.00

*Minimum radial strain attainable 230×10^{-6} in. per in.

[after Terrel and Monismith (70)]

Table 25. Gravel Equivalents of Structural Layers in Feet

Asphalt Concrete										BTB and LTB	Cement-treated Base			Aggregate base	Aggregate subbase
Traffic Index (TI)											Class				
5 and below	5.5 6.0	6.5 7.0	7.5 8.0	8.5 9.0	9.5 10.0	10.5 11.0	11.5 12.0	12.5 13.0	13.5 14.0		A	B	C		
Gravel Equivalent Factor (G_f)										G_f	G_f			G_f	G_f
2.50	2.32	2.14	2.01	1.89	1.79	1.71	1.64	1.57	1.52	1.2	1.7	1.5	1.2	1.1	1.0

Notes:

BTB is bituminous-treated base.
LTB is lime-treated base.
For the design of road-mixed asphalt surfacing, use 0.8 of the gravel equivalent factors (G_f) shown above for asphalt concrete.

[after reference (80)]

Table 26. Asphalt Institute's Layer Equivalencies

Material	Equivalency*
high quality untreated granular base	2.0
low quality untreated granular base	2.7
hot-mix sand asphalt base	1.3
liquid and emulsified asphalt bases	1.4

*Expressed in inches of stated material required to 1 inch of good quality asphalt concrete.

[after reference (81)]

Table 27. Oklahoma Layer Equivalencies

Material	Equivalency*, Inches
asphalt concrete	1.5
blended rock asphalt	1.5
black base	1.25
hot mix sand asphalt	1.0
soil asphalt	1.0
soil cement	1.0

*Expressed such that the stated material will replace indicated inches of stabilized aggregate base.

[after reference (82)]

TABLE 28

SUMMARY OF STRUCTURAL LAYER COEFFICIENTS USED FOR DIFFERENT PAVEMENT COMPONENTS

STATE	SURFACE COURSES: PLANT MIX (HIGH STABILITY)	ROAD MIX (LOW STABILITY)	OTHER		BASE COURSES: UNTREATED		CEMENT-TREATED		LIME-TREATED	BITUMINOUS-TREATED		SUBBASES	
Alabama	0.44	0.20	Sand asphalt	0.40	Limestone Slag Sandstone Granite	0.14 0.14 0.13 0.12	< 400 psi 400–650 psi > 650 psi	0.15 0.20 0.23	—	Coarse graded Sand	0.30 0.25	Sand & sandy clay Chert, low P.I. Topsoil Float gravel Sand & silty clay	0.11 0.10 0.09 0.09 0.05
Arizona	0.35–0.44	0.25–0.38	Sand asphalt	0.25	Sand & gravel, well graded Cinders Sandy gravel, mostly sand	0.14 0.12–0.14 0.11–0.13	< 300 psi 300–500 psi > 500 psi	0.15 0.18–0.25 0.25–0.30	—	Sand-gravel Sand	0.25–0.34 0.20	Sand-gravel, well graded Crushed stone or cinders Sand & silty clay	0.14 0.12 0.05–0.10
Delaware	0.35–0.40	—		—	Waterbound macadam Crusher run Quarry waste Select borrow	0.20 0.14 0.11 0.08	Soil-cement	0.20	—	Asph. stab.	0.10	Select borrow	0.08
Massachusetts	0.44	—		—	Crushed stone	0.14	—		—	Black base Penetrated crushed stone	0.34 0.29	Gravel Select material	0.11 0.08
Minnesota	0.315	—	Plant-mix sand asphalt (low stab.)	0.28	Crushed rock (Class 5 & 6 gravel) Sandy gravel	0.14 0.07	—		—		0.175–0.21	Sandy gravel (Cl.3 & 4 gravel) Selected granular (<12% minus #200)	0.105 0.07
Montana	0.30–0.35	0.20		—	Crushed gravel < 1½" > 1½" Select surf. Spec. borrow Sand	 0.14 0.12 0.10 0.07 0.05	< 400 psi > 400 psi	0.15 0.20	0.15	Plant mix Bit. stab.	0.25–0.30 0.20	—	
Nevada	0.30–0.35	0.17–0.25		—	Crushed gravel Crushed rock	0.10–0.12 0.13–0.16	—		—	Plant mix	0.25–0.34	Gravel type 1 Select material	0.09–0.11 0.05–0.09
New Hampshire	0.38	0.20	Sand asphalt	0.20	Crushed gravel Bank run gravel Crushed stone	0.10 0.07 0.14	Gravel	0.17	—	Bit. conc. Gravel	0.34 0.24	Sand-gravel	0.05
New Mexico	0.30–0.45	0.20	Plant-mix seal	0.25	Quarry rock Crushed rock	0.10–0.15 0.06–0.12	< 400 psi 400–650 psi > 650 psi	0.12 0.17 0.23	0.05–0.10	Plant mix Road mix	0.30 0.15	Aggregate Borrow	0.06–0.12 0.05–0.10
Ohio	0.40			—	Aggregate Waterbound macadam	0.14 0.14	—		—	—			0.11
Pennsylvania	0.44	0.20	Sand asphalt	0.35	Crushed stone Dense grade	0.14 0.18	Soil-cement Cement aggr. plant mix	0.20 0.30	Soil-lime 0.20	Soil-bit. Plant mix	0.20 0.30	Sand-gravel	0.11
South Carolina	0.40	—	A. C. binder sand asphalt	0.35	Crushed rock	0.14	—		—	Black base Sand	0.30 0.25	—	
South Dakota	0.36–0.42	—		—		0.11		0.20	0.18	Hot mix aggregate coarse sand fine sand Cold mix aggregate	 0.30 0.24 0.18 0.15	Untreated	0.10
Utah	0.40	0.20	Plant-mix seal	0.40		0.12	400–650 psi	0.20	—	Coarse graded	0.30	Sand-gravel Sand or sandy clay	0.10 0.06–0.10
Wisconsin	0.44	0.20	Sand asphalt	0.40	Crushed gravel Crushed stone Waterbound macadam Sand-gravel uncrushed	0.10 0.14 0.15–0.20 0.07	< 400 psi 400–650 psi > 650 psi	0.15 0.20 0.23	0.15–0.30	Coarse graded plant mix Sand plant mix	0.34 0.30	Sand-gravel	0.05–0.11
Wyoming	0.30–0.40	—	Inverted penetration	0.20–0.25		0.05–0.12		0.15–0.25	0.07–0.12	Plant mix Emulsion	0.20–0.30 0.12–0.20	Special borrow	0.05–0.12

Notes:
Consult *AASHO Interim Guide* (56, Table A 4-1) for values used by the following states:
1. Indiana, Iowa, New Jersey, Tennessee, and Puerto Rico—values as shown.
2. North Carolina and North Dakota—values as shown, except 0.40 for bituminous treated base.
3. Maine—values as shown, with some modifications.
4. Maryland substitution values for materials to replace design thickness of asphalt hot-mix are the AASHO structural coefficients expressed in layer thicknesses, in inches.

Table 29. Stiffness Coefficients for Asphalt Stabilized Materials

Location			Thickness of Material, Inches	Type of Material	Stiffness Coefficient		
District	Highway	County			Mean	Standard Deviation	No. of Readings
5	US87	Lubbock	6.25	ACP	0.99	0.27	14
	US87	Lubbock	6.25	ACP	1.06	0.25	14
			1.5	ACP			
	US87	Lynn	4.5	B.B.	1.16	0.15	9
			0.5	ST			
	US87	Lynn	4.0	B.B.	1.13	0.10	6
11	US69	Angelina	10.0	ACP	1.18	0.15	24
	US69	Angelina	10.0	ACP	1.21	0.22	49
15	IH35	Frio	10.0	B.B.	0.70	0.05	24
			10.0	B.B.			
	IH35	Frio	6.0	A.S.B.	0.52	0.03	24*
17	IH45	Walker	12.0	B.B.	0.77	0.09	27
			8.0	H.S.B.			
	IH45	Madison	4.0	A.S.B.	0.70	0.11	19
			8.0	H.S.B.			
	IH45	Madison	4.0	A.S.B.	0.87	0.11	21
	IH45	Walker	12.0	B.B.	0.65	0.08	25
			1.0	ACP			
	US290	Washington	5.0	B.B.	1.87	0.58	14
			1.0	ACP			
	US290	Washington	7.0	B.B.	1.43	0.30	21
19	IH30	Titus	8.0	B.B.	2.06	0.45	67
			8.0	ACP			
	SH98	Bowie	8.0	A.S.B.	0.48	0.01	5
			8.0	ACP			
	SH98	Bowie	8.0	A.S.B.	0.49	0.03	5
			4.0	ACP			
	SH98	Bowie	8.0	A.S.B.	0.47	0.13	14

ST- Surface treatment
ACP- Asphalt concrete pavement
B.B.- Black Base
A.S.B.- Road mixed asphalt stabilized base
H.S.B.- Hot mixed sand base

*Contains 6 inches of asphalt treated subgrade

Table 30. Layer Equivalencies as Determined by Texas Highway Department Flexible Pavement Design Methods*

Temperature Constant	Total Traffic, Eq. 18 Kip Axle Loads X 10^6	Subgrade Stiffness Coefficient			
		0.15	0.20	0.25	0.30
9	1	2.3	2.4	2.4	2.5
	3	2.6	2.8	2.9	3.1
	6	3.3	3.7	3.8	4.2
	10	---	---	---	---
25	1	2.2	2.3	2.4	---
	3	2.3	2.4	2.5	2.5
	6	2.5	2.6	2.7	2.8
	10	2.8	2.9	3.1	3.2
38	1	2.1	2.1	---	---
	3	2.3	2.3	2.3	---
	6	2.4	2.5	2.6	2.8
	10	2.5	2.7	2.8	3.1

*Layer Equivalencies assume the stiffness coefficient of untreated base is 0.50 and treated base is 1.00.

Table 31. Average Low Bid Prices - May 1974

MATERIAL	COST PER BID UNIT, DOLLARS	COST PER INCH OF DEPTH, DOLLARS
A. Flexible Base Course		
1. Caliche	6.50 per ton	0.35
2. Gravel	2.85 per cu. yd.	0.08
3. Iron Ore	3.05 per cu. yd.	0.09
4. Crushed Stone	2.85 per ton	0.16
5. Unspecified	6.50 per ton	0.35
B. Lime Stabilized Subgrade		
Lime	31.00 per ton	0.19
Lime Stabilization (6 in.)	1.07 per sq. yd.	
C. Cement Stabilization		
Cement	7.72 per bbl.	0.25
Cement Stabilization (6 in.)	1.42 per sq. yd.	
D. Cement Stabilized Base (6 in.)	5.70 per sq. yd.	0.95
E. Black Base		
Asphalt Cement	72.50 per ton	0.70
Aggregate	9.00 per ton	
F. Asphalt Concrete		
Asphalt Cement	53.20 per ton	1.10
Aggregate	16.50 per ton	

[after reference (88)]

Table 32. Typical Price Range - June 1974 Letting*

District	County	Item	Cost, Dollars Per Ton	Cost, Dollars Square Yard Inch of Depth
1	Hopkins	ACP	14 - 17	0.75 - 0.91
4	Armstrong	ACP	16.50 to 18.00	0.88 - 0.97
5	Cochran	U.B.	5.50 to 7.00	0.29 - 0.37
9	Hill	ACP	18.00 to 20.00	0.96 - 1.07
		B.B.	14.00 to 16.00	0.75 - 0.86
	McClennan	ACP	14.70 to 17.15	0.78 - 0.93
		B.B.	14.50 to 16.00	0.77 - 0.86
11	Polk	ACP	20.00 to 23.00	1.07 - 1.23
		B.B.	14.00 to 18.00	0.75 - 0.96
	Nacogdoches	ACP	20.00 to 25.00	1.07 - 1.34
		B.B.	14.00 to 16.00	0.75 - 0.86
12	Harris	ACP	20.50 to 24.10	1.10 - 1.30
		B.B.	19.00 to 21.00	1.02 - 1.13
		U.B.	17.00 to 20.00	0.91 - 1.07
15	Bexar	ACP	17.00 to 19.00	0.91 - 1.02
	Frio	ACP	15.00	0.80
		U.B.	8.00	0.43
16	Nueces	ACP	20.00 to 23.00	1.07 - 1.23
	San Patricio	ACP	18.00 to 30.00	0.96 - 1.60
		B.B.	16.00 to 20.00	0.86 - 1.13
		L.T.	---	0.14
21	Hidalgo	ACP	15.00 to 17.00	0.80 - 0.91
		L.T.		0.13 - 0.17
24	El Paso	ACP	19.50 to 22.00	1.05 - 1.18
	Hudspeth	U.B.	5.50 to 6.50	0.29 - 0.35
		ACP	11.00 to 14.00	0.59 - 0.75
		B.B.	10.30 to 13.25	0.56 - 0.72

*Costs selected for geographic location.

ACP - Asphalt Concrete Pavement   B.B. - Black Base
U.B. - Untreated Base   L.T. - Lime Treated Subgrade

[after reference (89)]

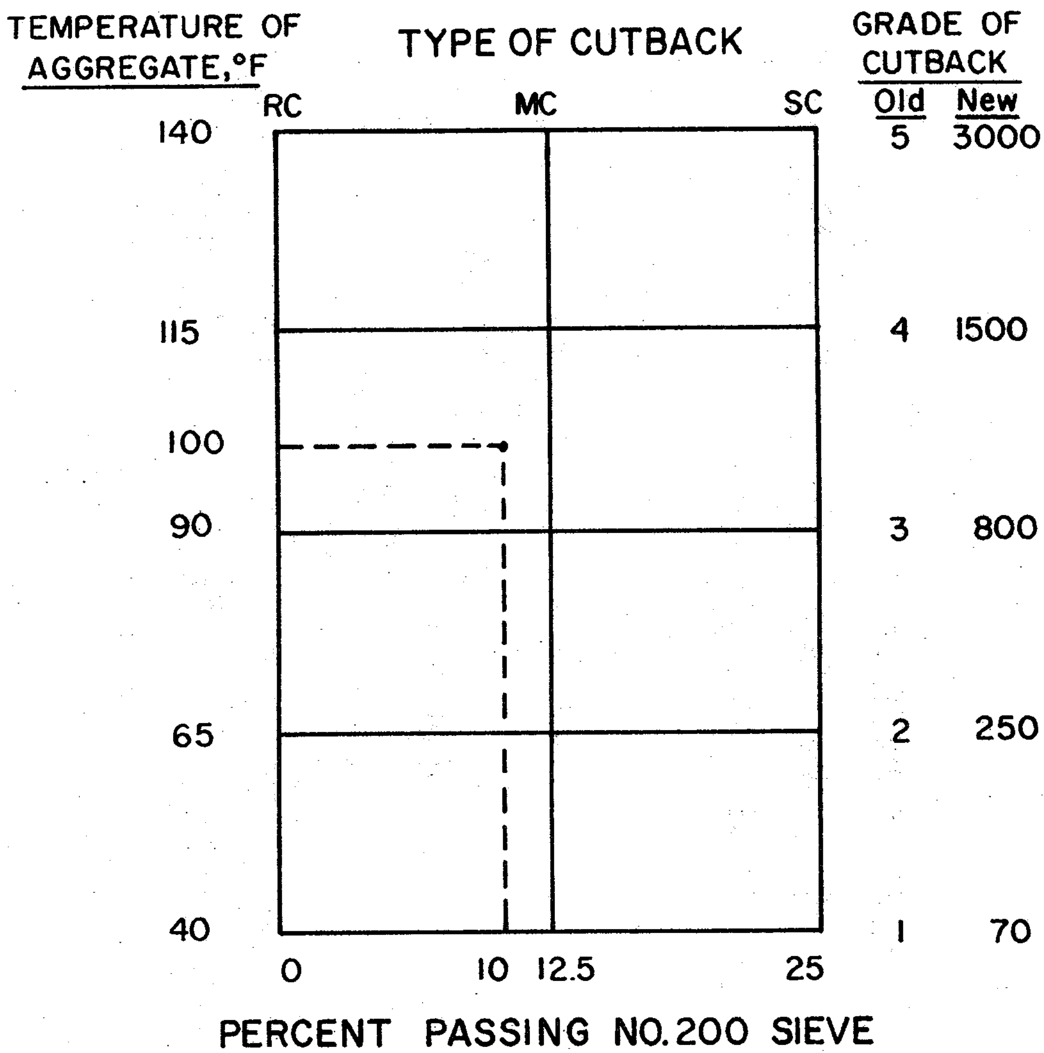
Table 33. Component Cost of Producing Hot Mixed Asphalt Concrete

Component Cost Item	Percent of In-Place Cost
Plant Labor	4.05
Plant Fuel	0.19
Plant Expense	15.06
Dryer Fuel	2.32
General Overhead	1.35
Laydown Cost	11.58
Materials (Aggregate and Asphalt)	50.97
Haul to Job	14.48

[after reference (87)]

Table 34. Characteristics of Selected Materials of Bituminous Stabilization

Sieve Sizes	20 - SH87 Jefferson Co. Shoulders	20 - FM255 Jasper Co. Shoulders	20 - US96 Jasper Stockpile (Plant Site)	20 - FM255 Newton Co. Pit #3	21 - Fardyce Co. Sullivan City	21 - LaJoya Crow Gravel Pit	21 - 6Mi.W. Mission Beck Pit (Caliche)	21 - 20 Mi. N. Edinburgh Guirra Pit	21 - 6 Mi. W. Mission Beck Pit (Sand-Gravel)	25 - FM3182 Wheeler Co. S. End Sweetwater Creek	25 - FM1046 Wheeler Co. 1 M. E. Briscoe	16 - PR.22 Kleberg Co. Padre Is. Dune Sand	16 - SH 75 Aransas Co. Rt. of way	13 - Eagle Lake (Screenings) Thors. Matls. Stockpile	13 - US59 Victoria Co. Colato Cr. Bed	11 - Angelina Co. Daniels Sand	11 - Angelina Co. Vincent Sand	11 - Trinity Co. Caliche #3736	11 - Lufkin Gibson Sand	5 - Lubbock Co. (10 ME) Long Property	5 - FM168 Lamb Co. S. of Olton	5 - R.C.D. 7 M. E. Lubbock Ransom Canyon Rd.
1"	---	---	---	---	---	---	17.0	17.1	---	---	---	---	---	---	---	---	---	---	---	---	---	6.8
3/4"	2.7	---	---	---	---	---	30.0	28.2	16.8	---	---	---	---	---	---	---	---	---	---	---	---	10.4
1/2"	3.7	---	---	---	---	---	44.7	34.6	28.0						---	---	---	---	---	---	---	21.7
3/8"	5.0	---	0.6	0	---	0.1	57.6	39.3	35.0	---	---	---	---	---	---	---	---	---	---	---	---	27.0
#4	12.5	---	1.4	0	---	2.5	62.9	51.0	50.3	---	.4	---	---	39.0	---	.5	.2	---	---	3.8	---	37.0
#8	19.3	.5	2.5	.05	.06	5.1	69.3	60.0	61.3	---	.6	---	---	89.5	---	1.0	.3	.2	.07	13.1	---	44.7
#10	20.4	.6	2.7	.07	.08	5.2	---	---	63.4	.06	2.1	---	---	92.5	.03	1.1	.3	.7	.08	14.8	---	46.0
#16	24.5	1.0	5.5	.2	.2	7.2	72.8	66.5	67.8	.3	4.0	.07	.4	96.0	.16	1.2	.4	11.5	0.1	20.2	.01	49.5
#30	29.1	2.9	17.8	2.2	.8	8.7	75.3	70.8	71.9	3.4	9.4	.2	.5	97.7	2.6	2.8	.7	56.2	0.8	24.9	0.1	52.6
#40	30.9	6.7	33.6	7.0	1.5	9.2	76.2	72.3	73.6	11.6	17.6	.4	.8	97.9	17.6	10.5	1.7	67.0	11.7	27.0	.3	53.8
#50	33.0	18.6	58.4	27.4	4.7	10.0	77.5	73.7	76.2	44.3	31.0	.5	1.2	98.3	58.2	34.0	4.9	73.6	49.3	30.0	2.8	54.9
#60	37.1	34.0	71.5	41.6	15.0	11.9	79.2	75.1	79.2	59.4	44.0	.8	2.0	98.5	84.8	57.5	10.0	77.8	73.1	33.5	26.0	56.4
#80	63.7	58.8	81.8	69.7	54.2	18.3	82.6	79.0	84.7	67.9	62.7	26.7	14.6	98.8	98.4	73.4	23.8	83.2	91.6	45.1	73.3	51.7
#100	73.6	67.1	83.8	75.0	68.9	22.3	84.2	82.0	87.0	76.8	69.8	58.9	48.9	98.9	99.2	76.4	31.6	84.9	94.0	51.6	86.3	55.[illegible]
#200	88.6	79.7	86.2	84.7	91.6	35.6	88.6	88.0	91.8	96.1	85.0	98.2	96.4	99.2	99.5	82.9	60.0	88.0	97.1	72.1	97.2	78.5
Sand Equivalent	38.8	18.0	18.0	31.5	51.3	4.3	26.2	38.0	46.5	41.3	33.0	97.6	84.6	100.0	96.5	28.8	11.0	71.7	57.5	18.0	41.0	23.0
Fines Modulus	1.92	.901	1.69	1.05	.747	.558	4.42	4.04	4.14	1.25	1.15	.597	.510	5.19	1.60	1.16	.381	2.26	1.44	1.44	.897	3.04
Plastic Index	0	0	7.8	0	0	13.7	2.4	14.5	0	0	0	0	0	0	0	0	0	0	0	4.7	0	7.1
Liquid Limit	22.5	20.3	22.8	18.3	23.0	30.8	23.3	37.4	24.5	20.3	21.5	24.8	25.1	---	23.5	18.4	25.7	13.2	23.8	27.0	21.0	27.2
Plastic Limit	NP	NP	15.0	NP	NP	17.1	21.3	14.5	NP	NP	NP	NP	NP	NP	NP	NP	NP	NP	NP	22.3	NP	20.1



Example: For aggregate temperature of 100°F and 10% passing #200 sieve, use MC 800 cutback.

Figure 1. Selection of type of cutback for stabilization. [after U. S. Navy (41)].

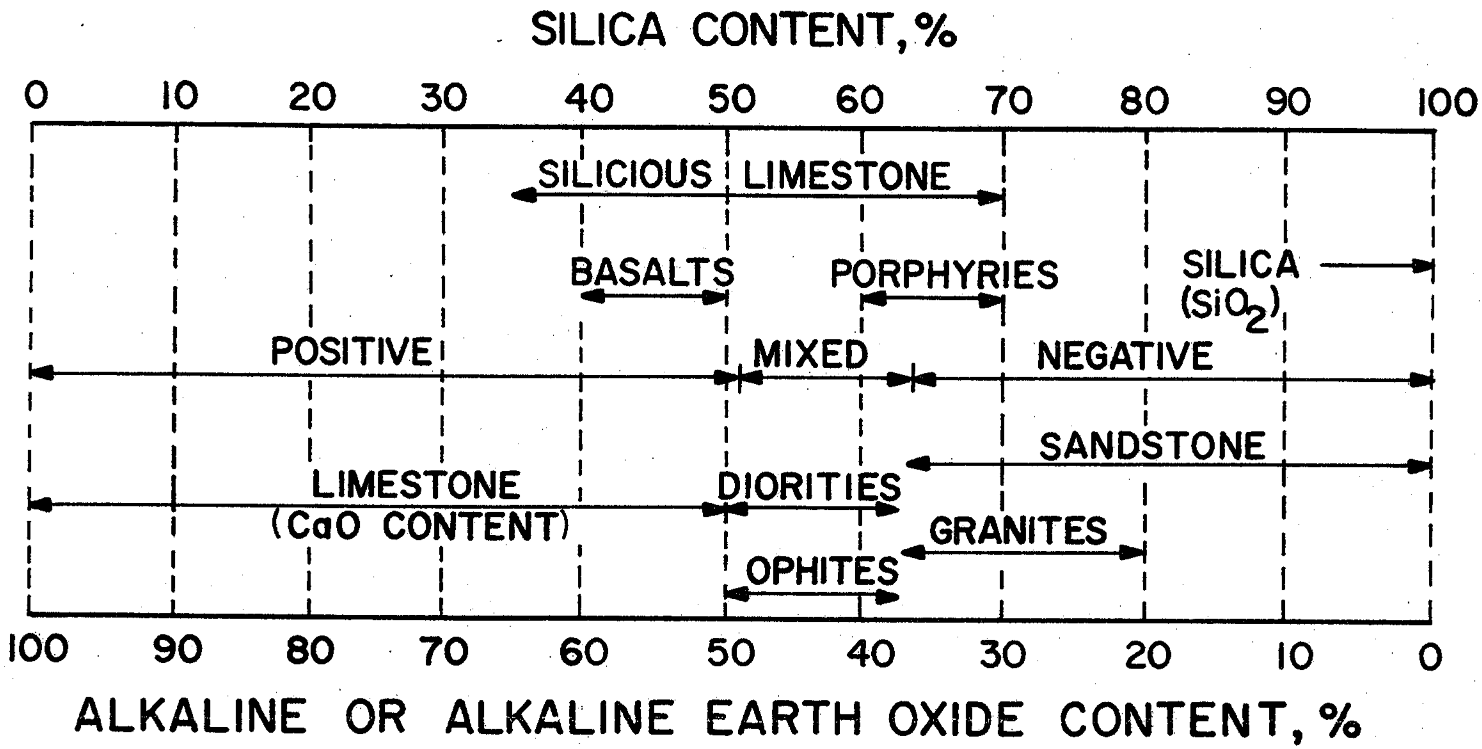


Figure 2. Classification of aggregates.

[after Mertens and Wright (54)]

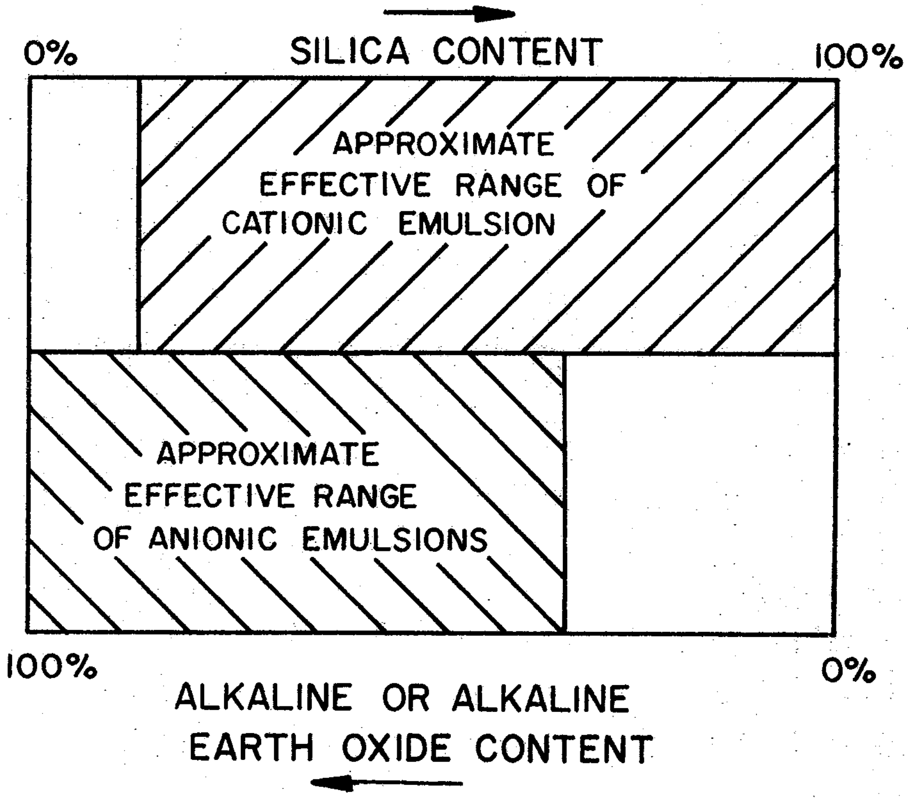


Figure 3. Approximate effective range of cationic and anionic emulsions on various types of aggregates.

[after Mertens and Wright (54)]

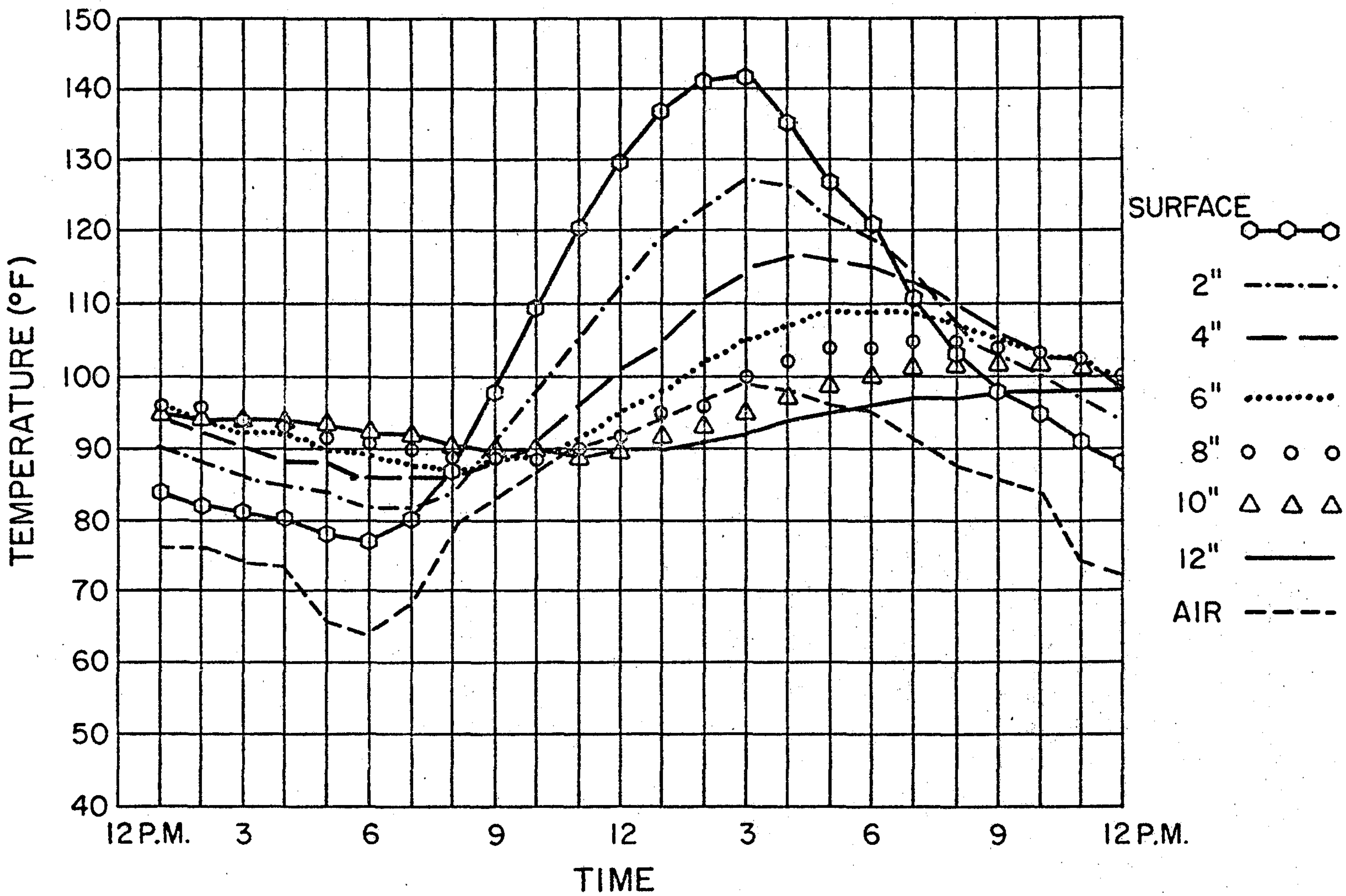


Figure 4. Asphalt-Concrete Pavement Temperatures on June 30, 1964.

[After Kallas (59)]

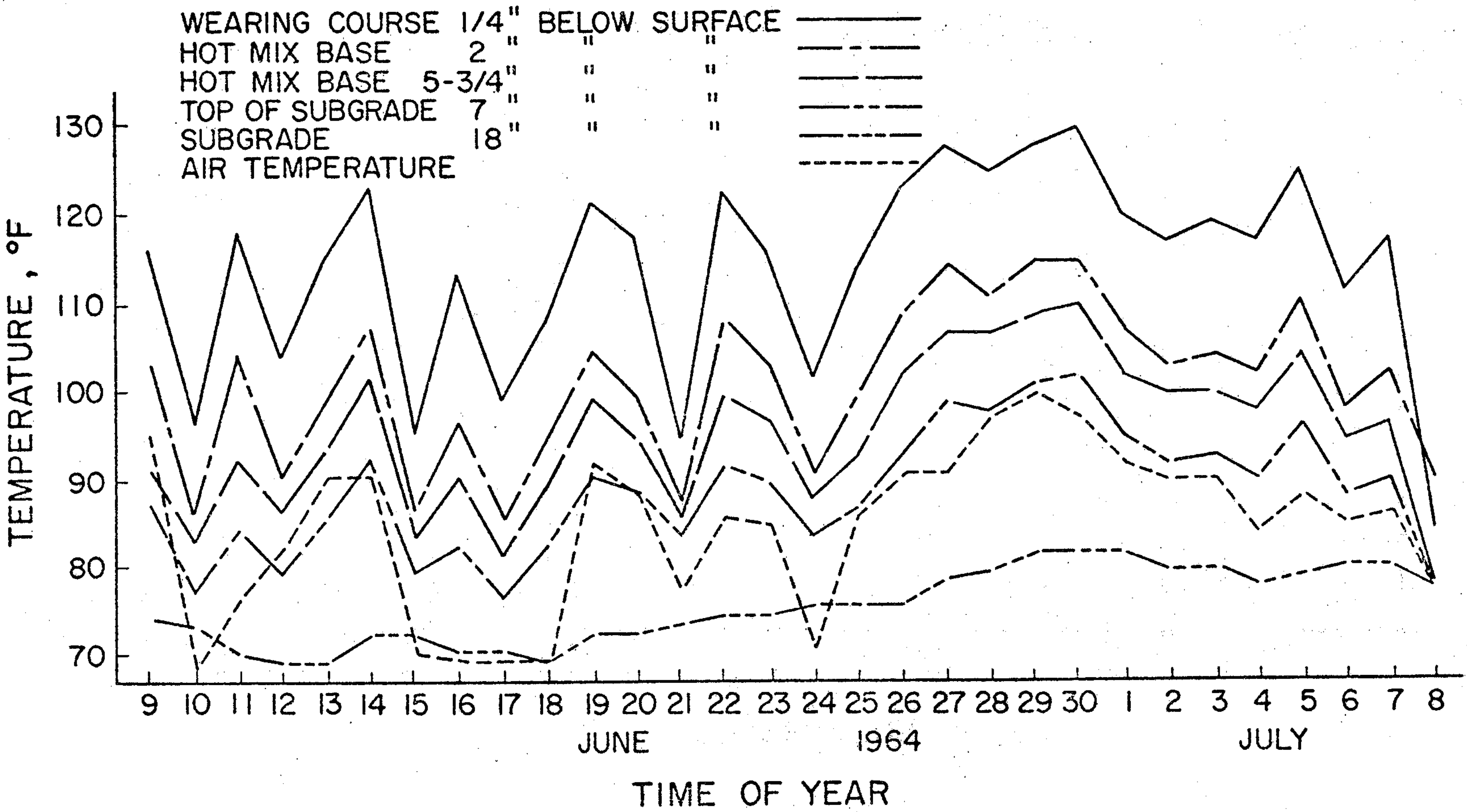


Figure 5. Asphalt Pavement Temperatures Wearing Course, Hot Mix Base and Subgrade.

[after Manz (60)]

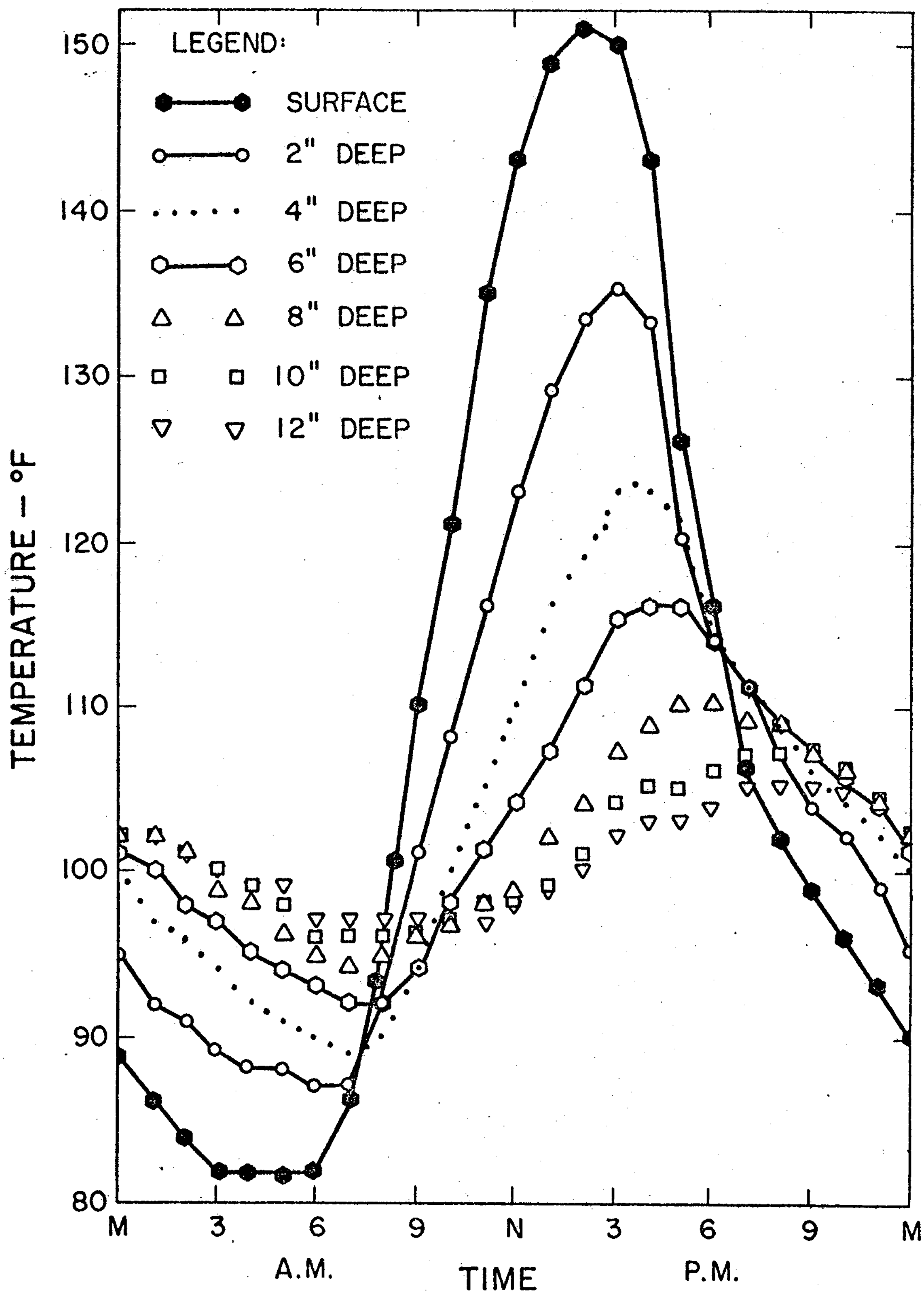


Figure 6. Typical Pavement Temperature Patterns in July.

[After Rumney and Jimenez (62)]

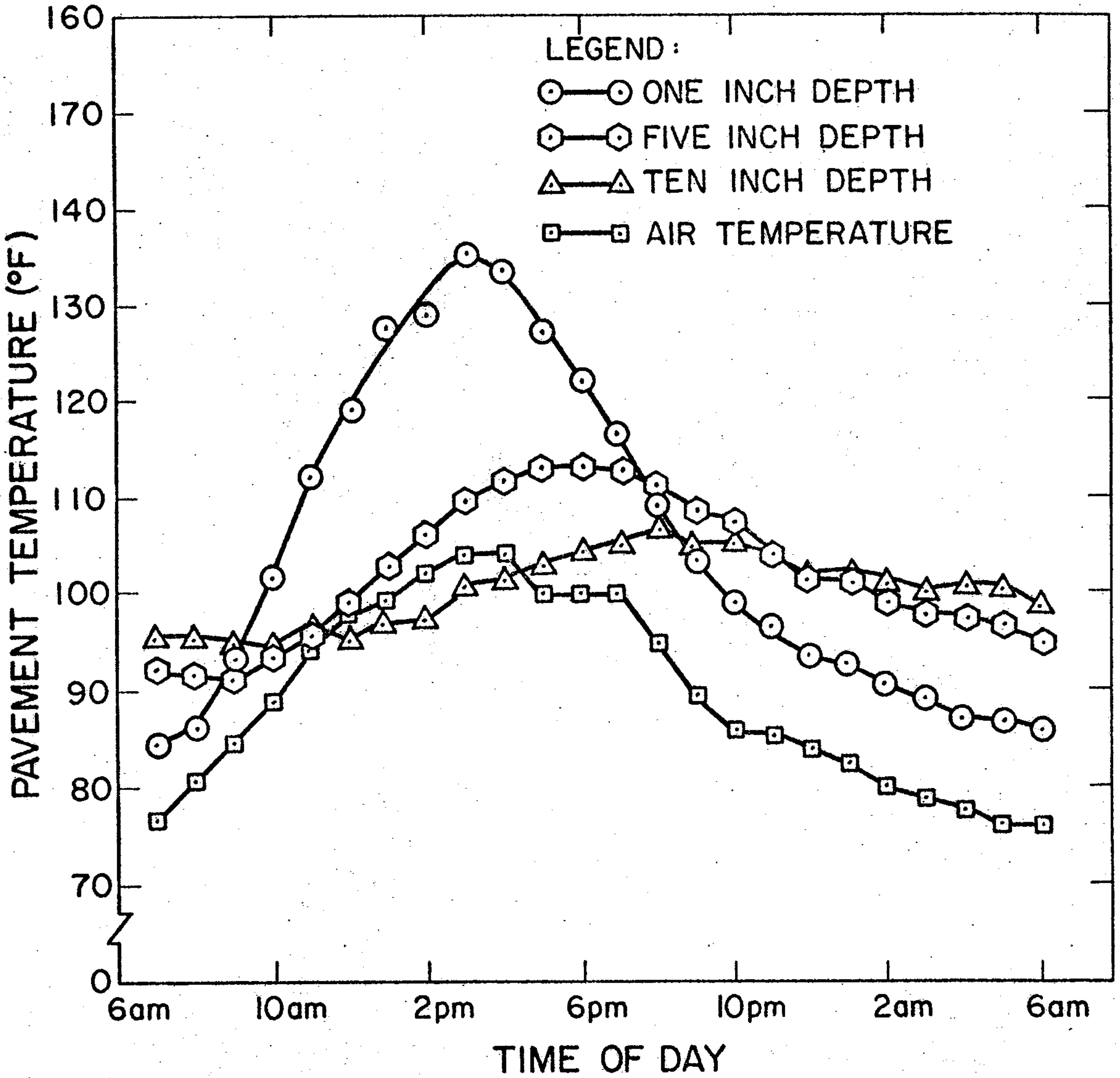


Figure 7. Pavement Temperatures in August Versus Time for Various Depths of Sandstone Mix.

[After Long (63)]

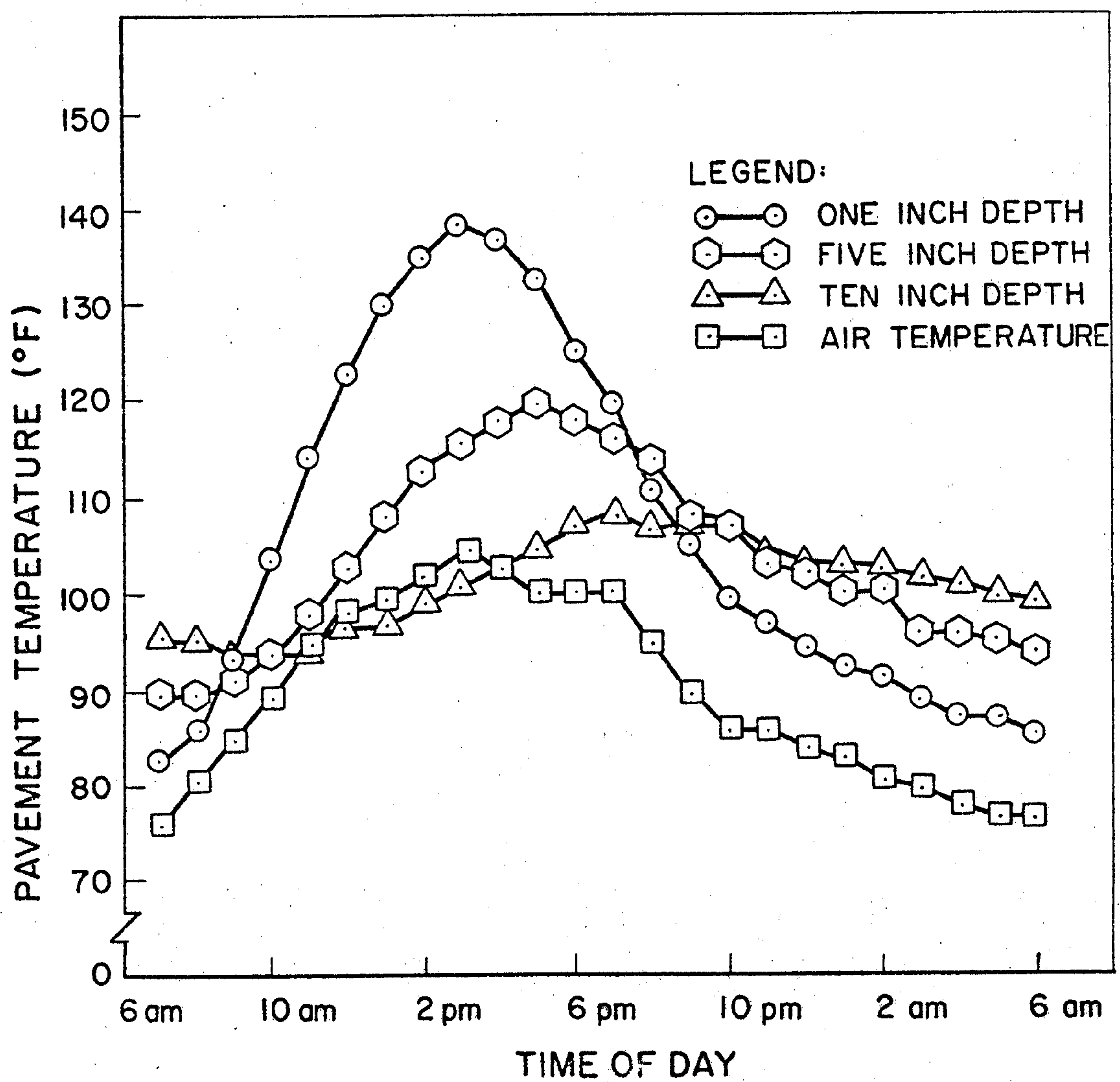
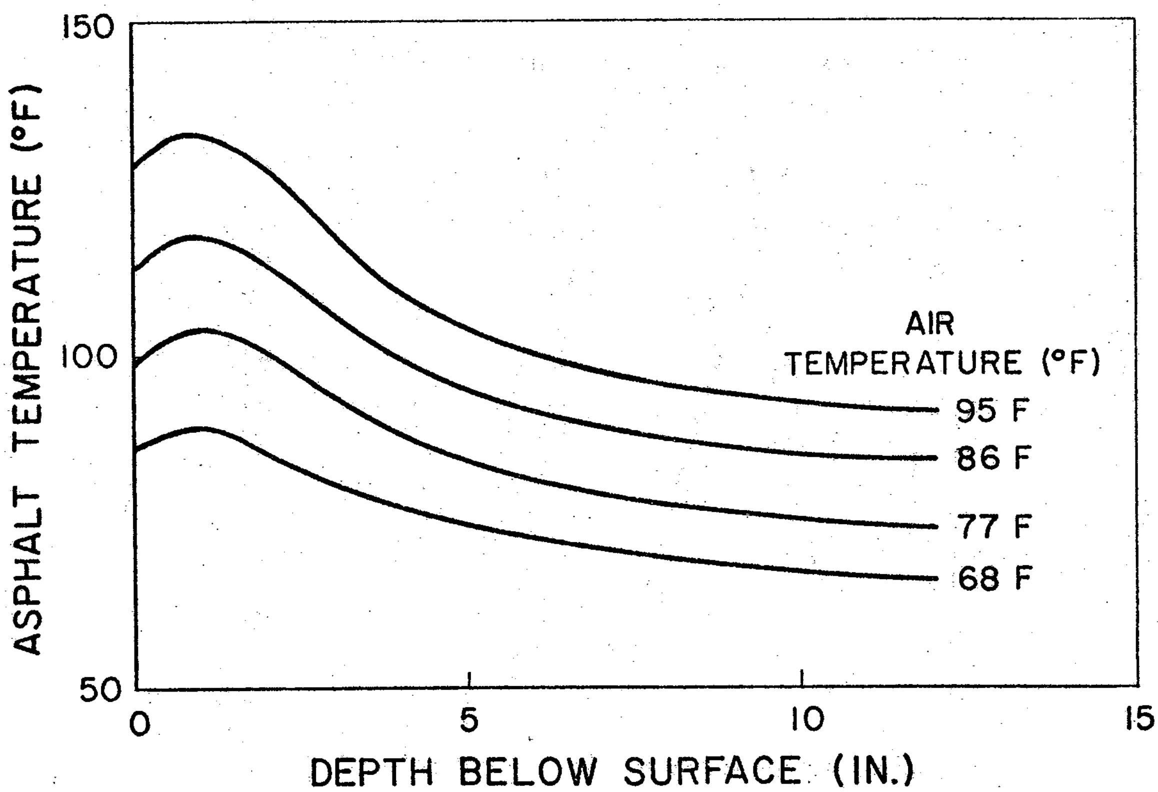


Figure 8. Pavement Temperatures in August Versus Time for Various Depths of Sand Mix.

[After Long (63)]



[After Dormon and Metcalf (68)]

Figure 9. Relation of Temperature of Asphalt Surface to Depth Below Surface.

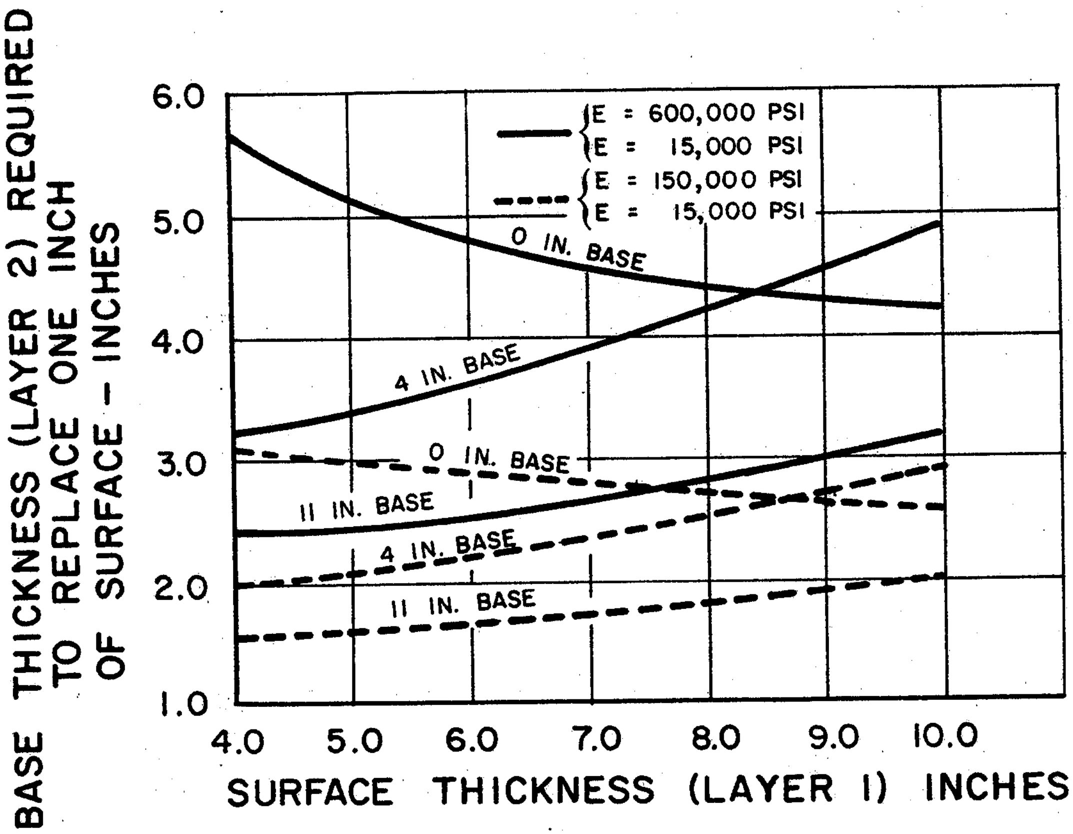


Figure 10. Typical Theoretical Equivalency Plot from Theoretical Vertical Stress on the Subgrade (Loop 4 Load Conditions).

[After Shook and Finn (71)]

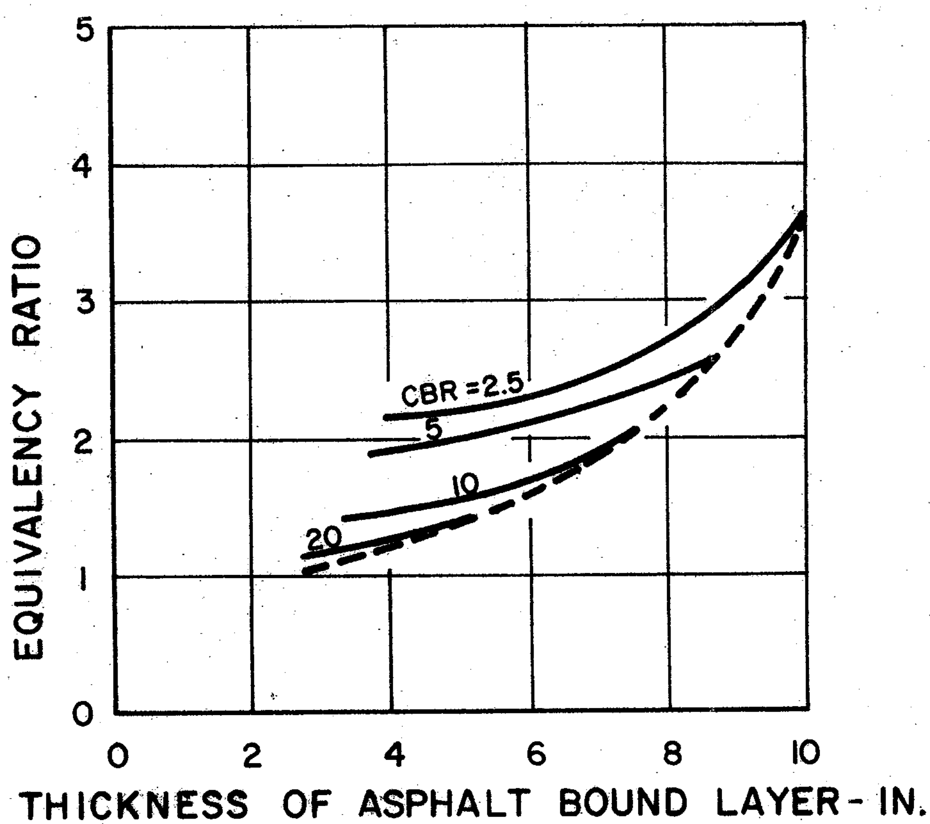


Figure 11. Relation of Equivalency Ratio to Thickness of Asphalt Bound Layer.

[After Lettier and Metcalf (72)]

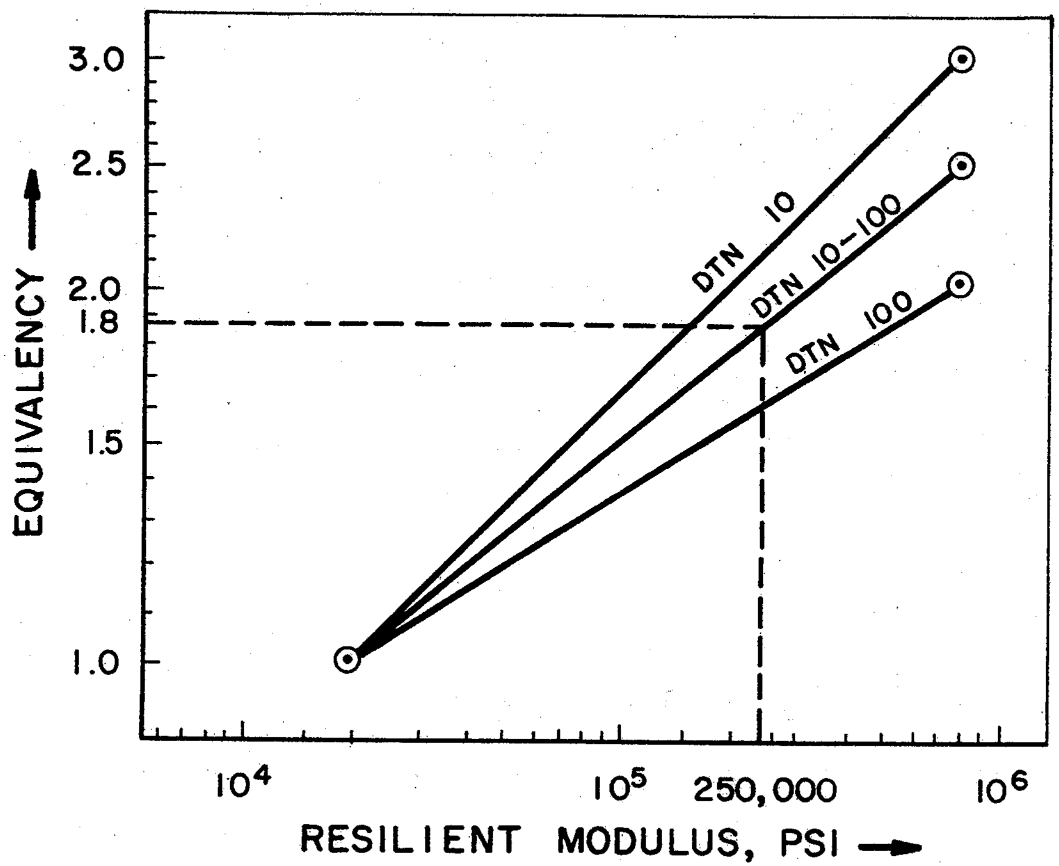


Figure 12. Chart for Estimated Equivalency.

[After Coyne (73)]

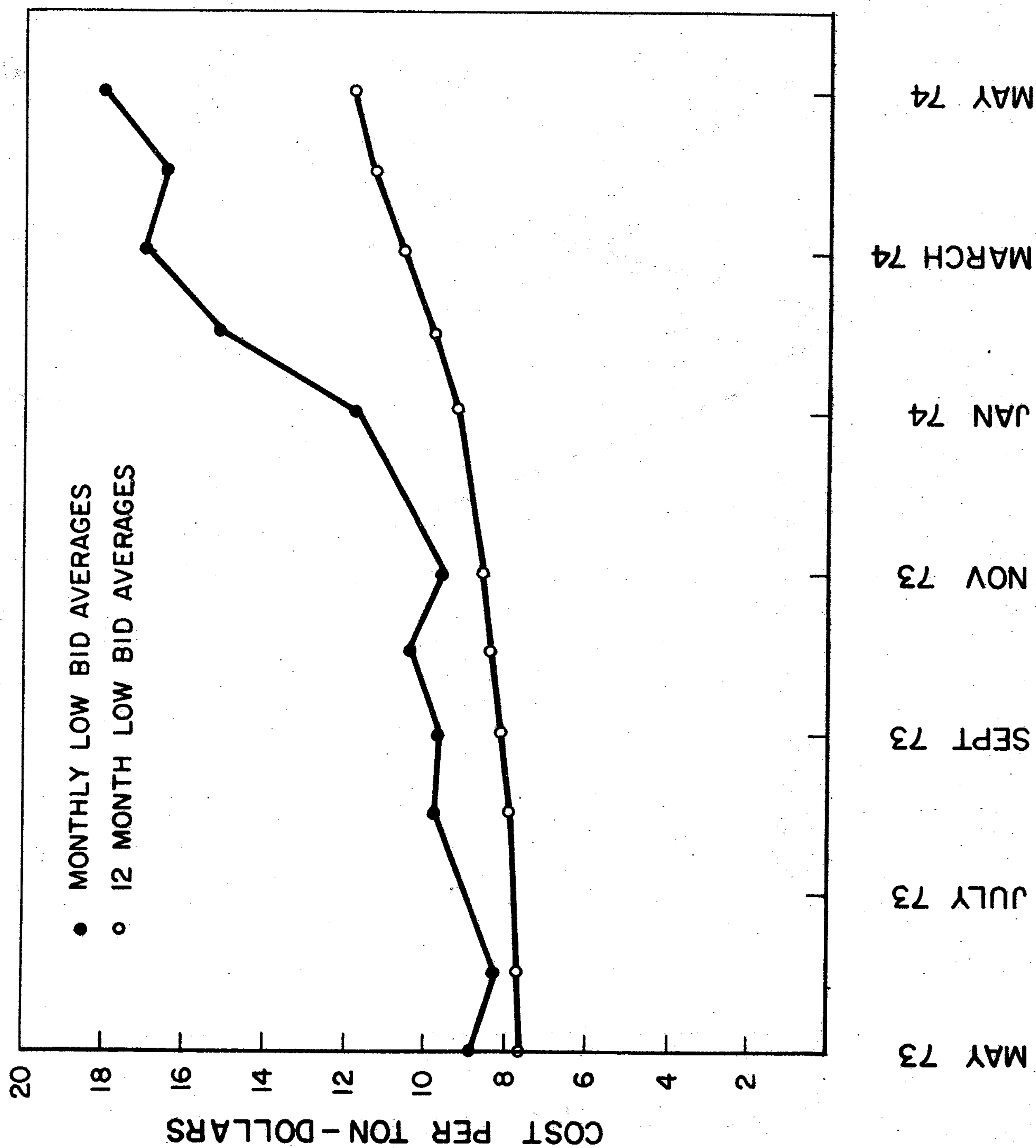


Figure 13. Black Base Cost Information.

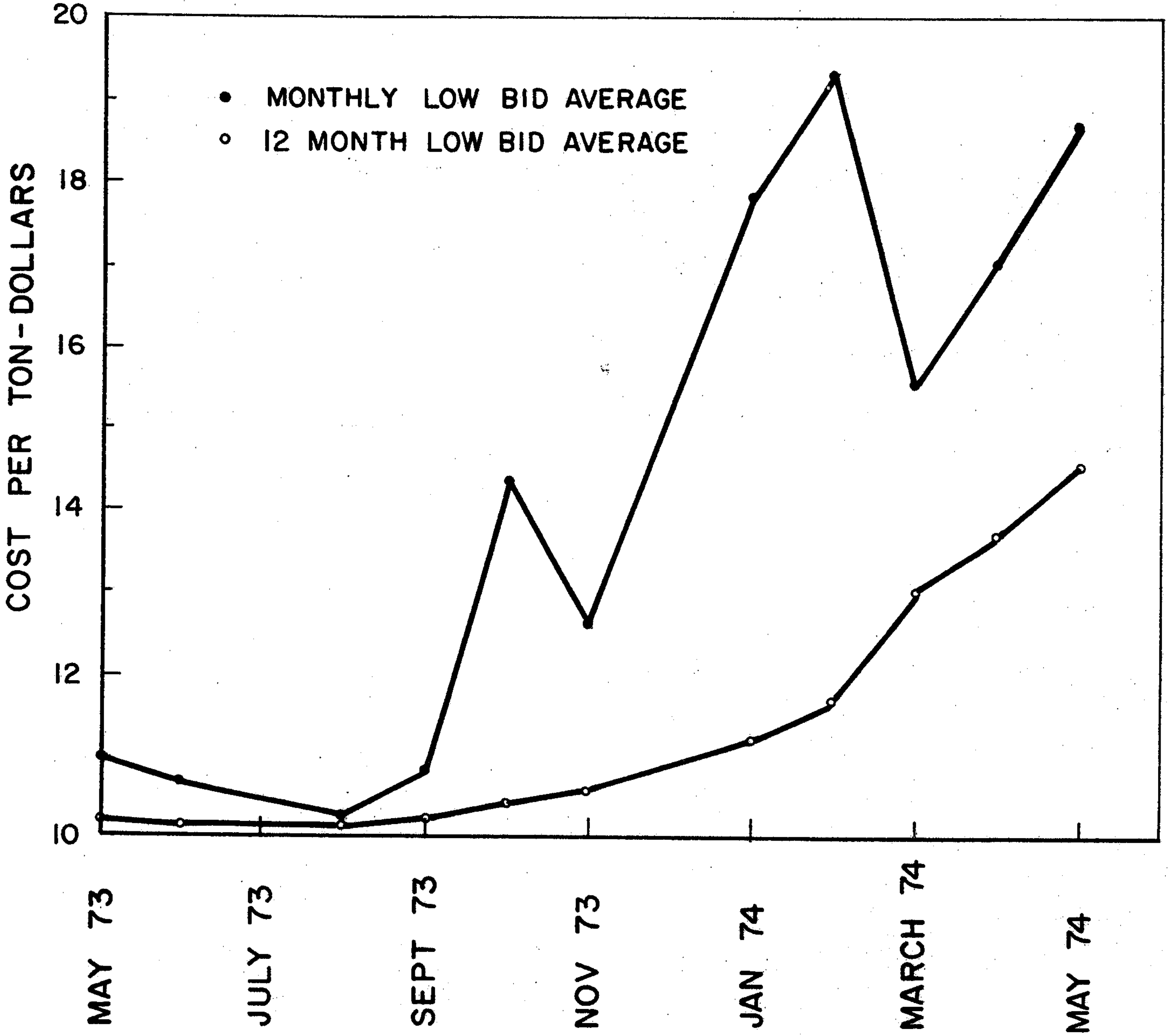


Figure 14. Asphalt Concrete Cost Information.

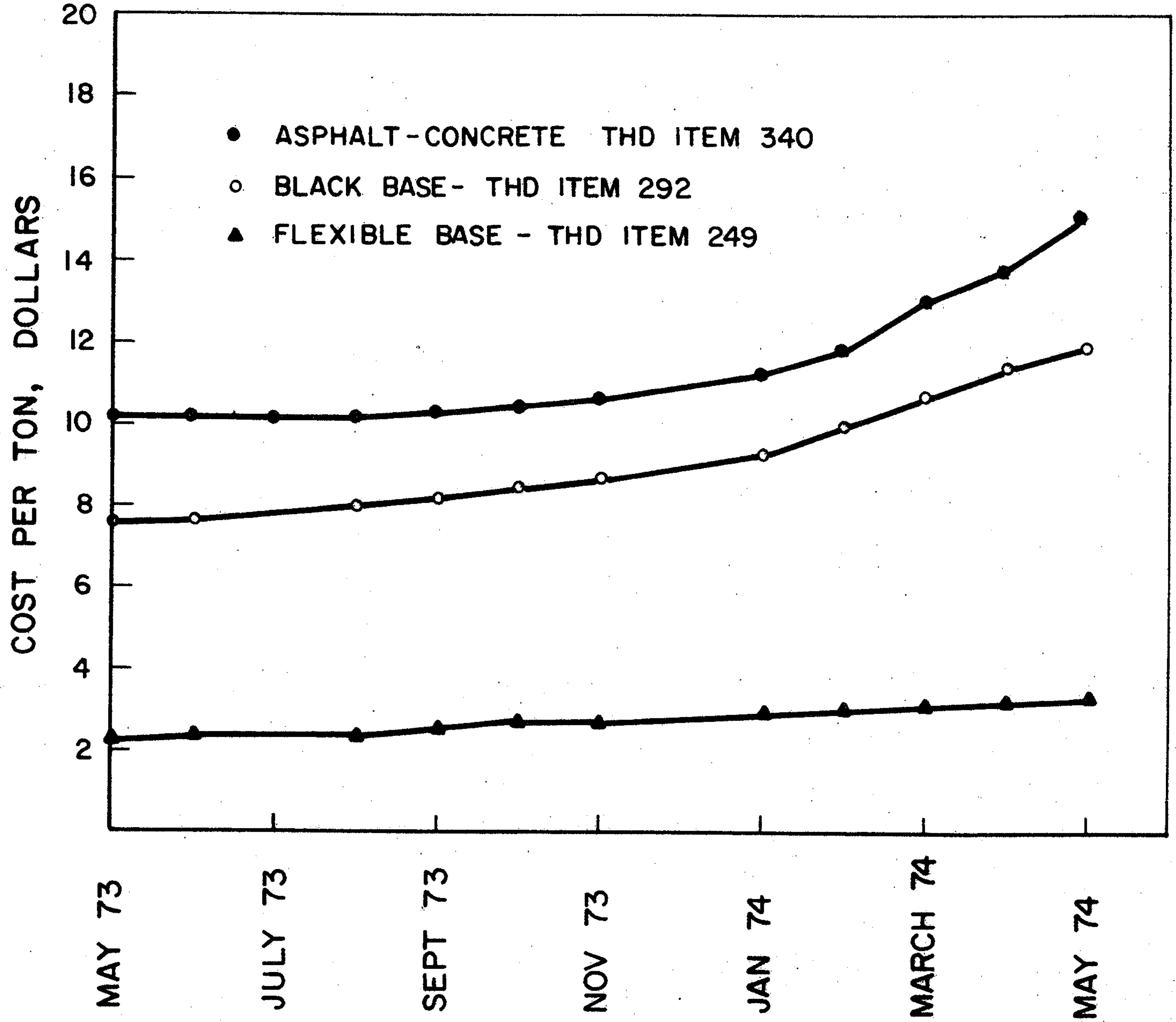


Figure 15. Base Course Cost Information.