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Symposium On Asphalt Paving Mixtures

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SYMPOSIA
DESIGNS
CONTROLS
ASPHALT
PAVEMENTS
STRUCTURES
FLEXIBLE

HIGHWAY RESEARCH BOARD

Research Report No. 7-B

SYMPOSIUM

INVESTIGATIONS OF THE DESIGN AND CONTROL OF ASPHALT PAVING MIXTURES AND THEIR ROLE IN THE STRUCTURAL DESIGN OF FLEXIBLE PAVEMENTS

BY

CORPS OF ENGINEERS - DEPARTMENT OF THE ARMY

PRESENTED AT THE TWENTY-EIGHTH ANNUAL MEETING

1948

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WASHINGTON 25, D. C. JUNE 1949

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INTRODUCTION

GAYLE McFADDEN[1] and WALTER C. RICKETTS[2]

During the latter part of November 1940, the responsibility for design and construction of military airfields and roads was assigned to the Corps of Engineers.

It soon became apparent for many reasons that it would be necessary (1) to select a simple testing device suitable for design and field control of asphalt pavements; (2) to correlate the results obtained by the use of the selected device with field performance for various wheel loads; (3) to establish suitable criteria for asphalt pavements; and (4) to establish the thicknesses of asphalt pavements of known quality for various wheel loads.

Among these reasons were (1) the difference in character and magnitude of wheel loads of airplanes and military vehicles as compared with highway traffic; (2) military airplanes with ever increasing wheel loads were being planned and constructed; (3) axle loads of numerous military vehicles exceeded those adopted and used in the design of public highways; (4) the usual methods employed for design and control of asphalt paving mixtures were not readily adaptable to the preparation and construction of asphalt pavements to meet the requirements in Continental United States and in theaters of operation for rapid design and field control; and (5) asphalt pavements of high quality, adequate for various wheel loads without overdesign, must be provided.

Asphalt paving mixtures were usually designed and constructed following certain criteria and specifications which had proved satisfactory over a period of time. Each engineer or designer had his own empirical design method and employed in some cases various types of testing machines to check design. In most instances there was little or no correlation between the design and field performance of the pavement under traffic and there was little experience or engineering data available in connection with very heavy wheel loads. The engineer adjusted the designed paving mixture at the start of construction based on his knowledge of materials and traffic conditions. The compaction of the pavement was often left to the judgement of the roller operator.

During World War II a different condition developed. It became necessary to expeditiously design and construct asphalt pavements for airfields and cantonment areas to carry loads far in excess of those carried by highways and city streets. Also, the methods of field control usually employed for pavement construction and plant control were not considered adequate on jobs which required the use of one or more asphalt plants producing 2500 tons or more of paving mixtures a day to meet completion dates.

A method was needed which could be stated in terms of definite procedures and criteria correlated with traffic so that asphalt pavements adequate for the design load could be consistently constructed by the field engineer.

To obtain the necessary data for establishing design procedure, pavement criteria and field control, investigation-

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al projects were initiated by the Office, Chief of Engineers. While the projects were directed toward the design of airfield pavements, the results obtained are equally applicable to the design of road and street pavements, particularly those which are to be subjected to considerable traffic of heavy vehicles.

INVESTIGATIONAL PROJECTS

The two initial projects consisting of a laboratory study and field investigation were authorized in September 1943 and in April 1944, respectively. These projects were assigned to the Corps of Engineers Flexible Pavement Laboratory, located at the U. S. Waterways Experiment Station, Vicksburg, Mississippi, for accomplishment. The overall objectives of the two initial projects were:

a. To select suitable test properties, develop laboratory techniques for their measurement, and by their use establish limiting criteria for satisfactory asphalt paving mixtures to meet the requirements of traffic for aircraft and heavy military vehicles.

b. To establish stability and thickness requirements of an asphalt pavement, adequate for the operation of single wheel loads of 15,000 and 37,000 lbs., and a dual wheel load of 60,000 lbs., when placed upon base course materials that range from low to high stability.

c. To fix gradation limits for asphaltic concrete, stone filled sand asphalt, and sand asphalt mixtures.

d. To investigate types of filler and select those considered best for asphalt paving mixtures.

e. To establish criteria for limiting the amount of filler that can be included in a satisfactory asphalt paving mixture for the herein specified use.

SCOPE

The scope of the overall investigation was to:

a. Study the existing methods of asphalt pavement design and select a method requiring testing apparatus adaptable to field design and control. Incorporate the selected testing apparatus with the military field CBR test kits, utilizing all the latter equipment, if practical, conduct a comprehensive laboratory investigation of the testing apparatus and perform such correlations with other existing apparatus as considered necessary or adviseable.

b. Conduct a comprehensive laboratory study utilizing the selected apparatus. Also compare the test properties of sand asphalt and asphaltic concrete mixtures in which such items as aggregate type and gradations and amount and type of filler are variables. Conduct studies to correlate laboratory compaction of asphalt paving mixtures with densities obtained in the field at the time of construction.

c. Construct a field test section of several qualities and thicknesses of asphalt pavements including surface treatments on three qualities of base and conduct accelerated traffic tests thereon.

d. Conduct final laboratory studies and analyze the data obtained from this investigation and establish satisfactory design criteria and control procedures.

LABORATORY STUDY

The first objective to be accomplished under the laboratory study was to select or develop, if necessary, a simple and highly portable testing machine. It was particularly desirable that the selected machine could be readily adapted to the existing California Bearing Ratio Testing equipment for field use by Engineer Troops.

A testing machine, which had been used by Mr. Bruce G. Marshall during his employment with the Mississippi State Highway Department, met the desired requirements and was tentatively selected. The final selection was to be based on the ability of the machine to satisfactorily measure properties of a paving mixture when compared to an existing machine known to be suitable for the purpose. Mr. Marshall, from whom the Marshall stability machine derived its name, was employed by the Flexible Pavement Laboratory during the initial laboratory study to further any desirable developments in the machine or test procedures.

A report of a study completed in 1943 by the Tulsa District of the Corps of Engineers of various types of testing machines in connection with the stability of rock asphalts and asphaltic concrete indicated that the Hubbard-Field machine was typical of those that satisfactorily measured the pertinent properties of an asphalt paving mixture.

Comparative laboratory tests were performed on a range of asphalt paving mixtures using both the Hubbard-Field and Marshall machines. This work indicated that the Marshall machine was satisfactory. The values obtained could be used for selecting proper asphalt content and reflected variations in gradation of aggregate, character of aggregate, variations in filler content, and penetration of asphalt.

The Marshall machine has been and is being used to measure the stability and flow of the paving mixtures used in connection with all phases of the overall investigation. However, it was necessary to redesign the compaction hammer and make revisions in the procedures originally used in designing paving mixtures by this method.

FIELD INVESTIGATIONS

In 1944 a test track was constructed on a well drained site at the Waterways Experiment Station, Vicksburg, Mississippi. The objectives of the test track were as follows:

a. To compare asphalt pavement mixtures which have a wide range of physical properties but equal stability values under traffic of 15,000, 37,000, and 60,000 lb. wheel loads (60,000 lb. load on dual tires).

b. To determine the stability values of asphalt pavement satisfactory for these wheel loads.

c. To establish the minimum thickness of asphalt pavements based on a specified stability for the three wheel loads outlined above on a high quality base course.

d. To determine the thickness of asphalt pavements of known stability necessary to support the wheel loads outlined above for bases of medium and low quality.

e. To compare the behavior of surface treatment on various type base courses under the three wheel loads.

f. To determine the relationship between the optimum asphalt content as determined by the Marshall method and the optimum required by traffic compaction of the three wheel loads listed above.

An additional objective formulated during the test program was to compare the effect of hot weather and cold weather traffic.

The reason for the selection of the above wheel loads was that in 1944 three types of airfields were being constructed based on their wheel loads.

TRAFFIC TESTS

Traffic testing was initiated on the 15,000 lb. lane in May 1945 and 3500 coverages were applied as of October 1945. Tests with the 37,000 lb. single wheel load were started in September 1945. Fifteen hundred coverages were applied to the 37,000 lb. lane in 1945 and summer of 1946. Tests using dual wheels (B-29 airplane) installed in the special constructed testing device and loaded to 60,000 lbs. were started in August 1945. Fifteen hundred coverages were applied to the 60,000 lb. lane in 1945 and summer of 1946. A coverage is defined as one load application over every point in a given area.

All traffic testing was conducted when the pavement temperature was 90 F. or higher. At times temperatures as high as 140 F. were recorded and most of the testing was conducted at temperatures ranging from 100 F. to 125 F. Some limited tests were conducted at much lower temperatures for comparison with the effect of hot weather traffic.

The experience obtained from the traffic testing indicated that 1500 coverages were sufficient for the purpose of the investigation. The physical properties did not change in the unfailed pavement by the application of additional coverages and as in the case of all accelerated tests, it is impossible to evaluate the effect of weather and time.

The tracking pattern was so arranged

as to eliminate effect of repeated coverage of one wheel on the same area. The direction of the traffic was reversed at regular intervals.

Pavement cores were taken at frequent intervals during the testing for laboratory examination and an accurate record supplemented with photographs was kept of all unusual conditions that developed in the pavements.

PAVEMENT CRITERIA

Using the data obtained from the laboratory study and the accelerated traffic tests, the following criteria have been established for asphaltic concrete surface course pavements for wheel loads between 15,000 lbs. and 37,000 lbs. on single wheels and 60,000 lbs. on dual wheels with gross tire pressures between 55 and 100 lbs. per sq. in. and net pressures as high as 140 lbs. per sq. inch.

Stability minimum	500 lbs.
Flow maximum	20
Percent voids, total mix	3 to 5
Percent voids, filled with asphalt	75 to 85

The pavement thicknesses conforming with the above criteria and considered suitable based on this investigation for the wheel loads used in the traffic testing, when placed on base courses of 80 CBR values or better, are as follows:

Wheel Loads Pounds	Total Pavement Thickness-Inches	Binder Course Thickness-Inches	Surface Course Thickness-Inches
15,000	2	-	2
37,000	3	1-½	1-½
60,000 (dual wheels)	3	1-½	1-½

As definite values were secured for mixes determined to be both on the rich and lean side of the optimum asphalt content, the above criteria are considered to be entirely valid. The criteria for sand asphalt, which will be discussed in a later paper, are considered to be less valid since mixes from the turnaround sections of the test track are included in the analysis and in general the sand asphalt mixes were on the rich side of optimum asphalt content.

All binder courses in asphaltic concrete sections of the test track performed in a satisfactory manner. Criteria were not established for binder courses as the present mold is too small to properly measure mixtures containing aggregate in excess of one-inch maximum.

Flow is the only property used in the criteria established for asphaltic concrete pavements suitable for heavy wheel loads which has not been generally used in connection with evaluating an asphalt pavement. It may be generally defined as the plasticity of the compressed mixture. However, the numerical values of flow do not vary a great deal until the mixture contains an excess of asphalt. Rich mixtures have higher flow values and the limiting values of flow have been established for satisfactory surface courses based on the results of the accelerated traffic testing with various wheel loads.

DESCRIPTION OF PAPERS

The papers composing this symposium were prepared with the view of making available in condensed form investigational data obtained to date which may be useful to the engineer engaged in the design and construction of roads or streets.

No attempt has been made to present all of the supporting data used in the development of the pavement criteria and the proper application of the Marshall machine to the design and control of asphalt paving mixtures. However, the comprehensive published report entitled, "Investigation of the Design and Control of Asphalt Paving Mixtures", (*1*)[1] consisting of three volumes, which contains all data, may be obtained from the Flexible

[1]Italicized figures in parentheses refer to the list of references at the end of the paper.

Pavement Laboratory, Waterways Experiment Station, Vicksburg, Mississippi.

The purpose of this paper is to briefly (1) review the activities of the Corps of Engineers in pavement design; (2) describe how the investigation fits into the over-all picture; (3) summarize the work completed; and (4) outline plans for future work.

In Paper No. 2, the preliminary analysis made to select test apparatus is summarized and in Paper No. 3, the results of the first laboratory study are reviewed. This study developed information on how various factors affect the characteristics of a paving mixture and resulted in a feasible design and control method based on the use of indicator tests. In Paper No. 4, the construction and testing of a field test section, together with analysis of the data, are presented. The purpose of the test section was to develop the design criteria for the proposed method. Paper No. 5 presents the adjustments that were made in the laboratory test procedures to insure that specimens on which designs would be based would closely approach field conditions. The final detailed test procedures for the method are presented in Paper No. 6. Paper No. 7 shows how the method is used in the actual design and construction of a pavement and the closing paper discusses the design as related to other features of a flexible pavement.

ADDITIONAL INVESTIGATIONAL WORK

Generally as an investigation progresses many items develop which merit further work. This investigation is no exception in this respect.

The present Marshall machine and procedures are applicable for conducting tests on paving mixtures containing aggregates of one-inch maximum size or less. Laboratory and field work are in progress to develop a mold and procedures suitable for use with the present Marshall machine capable of testing mixtures containing aggregates up to 2½-inch maximum size. The larger mold will be used primarily to establish test properties of asphaltic binder and base courses suitable for heavy wheel loads.

There are many opinions as to the exact percentage of material passing the No. 200 sieve that should be used to produce the most satisfactory pavements. Experience indicates that pavements containing excessive amounts of No. 200 mesh material tend to crack with age. The need to establish the most suitable percentages of No. 200 mesh material to use in producing pavements with long life as required for military installations is apparent.

The percentage of No. 200 mesh material in a runway pavement is particularly critical as the middle third of a runway receives most of the traffic. This leaves considerable areas which do not receive the beneficial kneading of concentrated traffic and may crack if not properly designed.

Some laboratory work has been accomplished towards the development of a machine that will measure the flexibility of an asphalt pavement. It is hoped that, with data obtained by the use of a flexibility machine, the most satisfactory limits for material passing No. 200 sieve, may be definitely established.

In order to further reduce the personal equation, decrease the amount of labor, and possibly cut the time required to produce a specimen, a machine for mechanically compacting test specimens is being developed. Laboratory work to date indicates such a machine will be available for use in the near future.

Since the start of this investigation, airplanes with wheel loads up to 150,000 lbs. have been designed and constructed and tires capable of being inflated to pressures up to 300 lbs. are distinct possibilities as standard equipment for future airplanes.

It is not known at this time whether or not the pavement criteria established as a result of the work performed in connection with this investigation are suitable for use in constructing pavements to be subjected to wheel loads in excess of 60,000 lbs. on dual wheels. The results obtained on a test track constructed and tested at Stockton, California may preclude the necessity of further investigational work to establish criteria

suitable for pavements subjected to very heavy wheel loads. The Stockton test track consists of sections of flexible pavements of varying thicknesses and has been subjected to the accelerated traffic of wheel loads of 150,000 lbs. and greater.

It is to be pointed out that asphalt cement has been used as the binding agent in all the work completed to date. It is considered that the methods and probably the design criteria with some modifications are also applicable to the harder grades of tar. It is contemplated that a study of hot mix tar concrete will be initiated in the near future. It is also believed that with modification of the procedures, the method can be used for the design of paving mixtures using cut-backs or other liquid types of bituminous binders.

CONCLUSIONS

It is considered that based on the data presented in this symposium, the conclusions noted below are justified:

a. The Marshall machine measures in a satisfactory manner characteristics pertinent to an asphalt pavement, reflecting changes in type of aggregate, asphalt content, penetration of asphalt, plasticity, quantity of No. 200 mesh material and gradation.

b. The design procedures permit the determination of the "optimum" asphalt content for a given mixture.

c. The Marshall machine and procedures are equally adaptable to design and field control of asphalt paving mixtures.

d. The design criteria for asphalt pavements, particularly asphaltic concrete, has been validated by traffic testing and laboratory data.

e. The thickness of asphalt pavement of known characteristics required for wheel loads up to and including 60,000 lbs. (dual wheels) on base courses of varying quality are valid as the values were established as the result of traffic tests on actual pavements.

f. The use of the highly portable Marshall machine and the development of comparatively simple procedures permits adequate asphalt pavements to be designed and field controlled in an expeditious manner, a prime requisite in connection with the construction of military installations.

Detailed procedures for the use of the Marshall machine for the design and field control of asphaltic concrete paving mixtures are now being prepared for publication in the Engineering Manual of the Corps of Engineers, Department of the Army.

ACKNOWLEDGEMENTS

The work reported in this symposium represents the combined efforts of the engineers and laboratory technicians of the Flexible Pavement Laboratory, Messrs. W. J. Turnbull and S. M. Fergus, Waterways Experiment Station and the authors of this paper.

Mr. A. L. Beller, formerly with the Flexible Pavement Laboratory, was in direct Charge of the mixture design, construction of test section and a large portion of the traffic testing.

The Engineer Board (now Engineer Research and Development Laboratories) participated in the initial laboratory work and the construction of the test section.

The following have reviewed the analysis made of the investigational data in the capacity of consultants: Professor W. J. Emmons, Messrs. H. W. Skidmore, J. L. Land, O. J. Porter, and A. H. Benedict.

SELECTION OF TEST EQUIPMENT

by JOHN M. GRIFFITH*

The first phase of the general project "Investigation of the Design and Control of Asphalt Paving Mixtures" *(1)* was the selection or development of a simple method of asphalt pavement design and control which would utilize easily portable testing apparatus that could be used in the field. It was particularly desired that the apparatus be adaptable to the California Bearing Ratio (CBR) testing equipment which was available to Corps of Engineers troops. This paper covers the primary aspects of the investigation made to select a method of design and control which would fit these requirements.

TULSA REPORT

Prior to the initiation of the investigation described in this symposium, the Tulsa District, Corps of Engineers, conducted a comprehensive laboratory investigation which is summarized in an unpublished report prepared by that office and titled "Comparative Laboratory Tests on Rock Asphalts and Hot-Mix Asphaltic Concrete Surfacing Materials." *(2)* Included in this report was a comparative study of the relative merits of four test methods which were most widely in use at that time. Comparative tests indicated that the Hubbard-Field test was the most satisfactory method of the four for general utility.

SELECTION OF MARSHALL EQUIPMENT

The results of the Tulsa investigation were studied, and their conclusions appeared to be reasonable based on their data. However, other factors had to be considered in the selection of test equipment to meet the requirements of the Corps of Engineers. In addition to selecting or devising a test method which was reliable and sensitive to the various factors entering into the design of asphalt pavements, it was also considered that the test equipment should be adaptable to the CBR test apparatus and that it should be easily portable. The available test equipment most nearly conforming to these latter requirements was that which had been devised by Bruce G. Marshall while working with the Mississippi State Highway Department. The Marshall stability equipment, however, had not been included in the Tulsa investigation previously referenced. In order to determine the over-all adequacy of the Marshall equipment in the design of asphaltic pavements the decision was made to conduct a series of comparative tests using both the Marshall and the Hubbard-Field equipment. The Hubbard-Field equipment was chosen for these comparative tests on the basis of the data contained in the Tulsa report and because it was one of the most widely used methods of asphalt pavement design at that time.

A detailed description of the Hubbard-Field method and apparatus may be found in a publication by the Asphalt Institute titled "The Rational Design of Asphalt Paving Mixtures." *(3)*

The test apparatus required for the Marshall test is relatively simple and compact. Figure 1 shows a view of the testing machine and the Marshall test head as developed at the start of the investigation by the Flexible Pavement Laboratory of the Waterways Experiment Station. Figure 2 shows the original adaptation of the Marshall test apparatus to the CBR testing frame furnished to troops.

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The sample of asphaltic mixture to be tested by the Marshall method was prepared by a standard compaction procedure in a 4-in. diameter mold to a height of 2½ in. This procedure consisted of compacting the specimens on one side only by 15 blows of a 10-lb. hammer falling 18 in. on a 2-in. diameter foot, followed by a 5000-lb. static leveling load applied over the surface of the specimen. The prepared sample is inserted into the Marshall test head (see Figure 3) after being heated in a hot water bath to 140 F., and the load is applied to the peripheral area of the specimen. The stability of a specimen is the maximum load in pounds which the compacted specimen will withstand. Load is applied to the test head by means of a mechanical jack at a rate of 2 in. per minute. The load is measured by means of a calibrated proving ring.

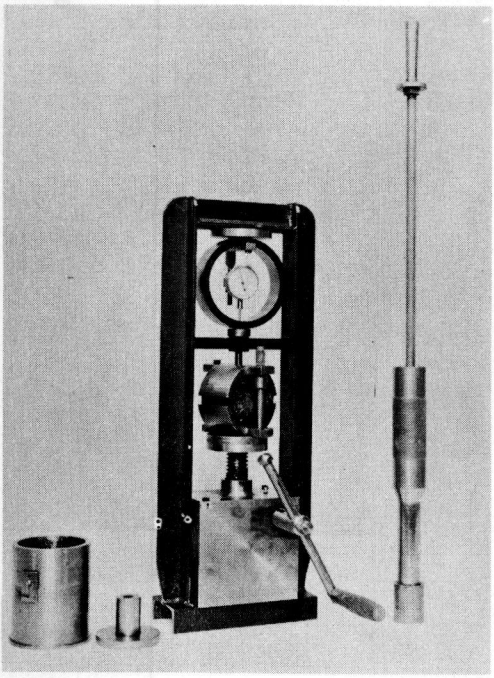


Figure 1. Marshall Testing Machine and Compaction Equipment Available in Field CBR Testing Kits

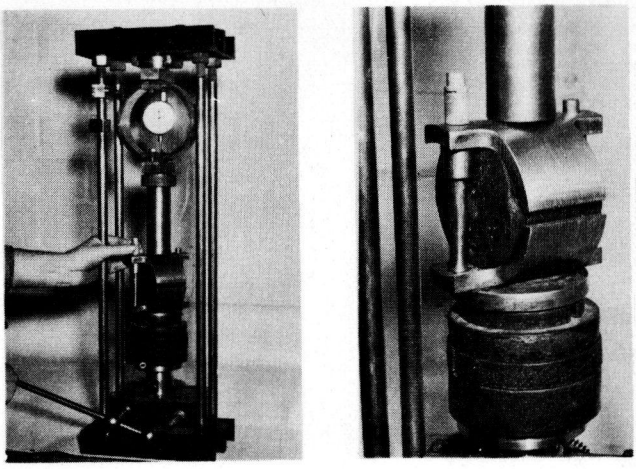
DEVELOPMENT OF FLOW METER

Prior to the initiation of the comparative laboratory test series using the Marshall and Hubbard-Field equipment it was recognized that some device for the measurement of strain of the test specimen would probably be a valuable addition to the Marshall stability test. Accordingly, a device named the "flow meter" was originated. The flow meter measures the total amount of movement between the two halves of the compression ring, or Marshall test head, as the specimen is failed. The operating principle of the flow meter may be observed by reference to Figure 3. The flow meter is a device consisting of a sleeve within which there is a cylinder graduated vertically on its side in units of one-hundredth of an inch. The internal cylinder fits snugly into the sleeve so that slight pressure is required to move the cylinder with respect to the sleeve. By means of the flow meter the strain occurring within the test specimen between no load and maximum load (Marshall stability) is determined.

COMPARISON OF HUBBARD-FIELD AND MARSHALL TEST APPARATUS

In the comparative test series three primary variables were introduced into the specimens compacted and tested by the two methods under study. In one series of tests the gradation of the aggregate blends used was varied from mixtures containing only 30 percent of gravel (material coarser than No. 10 size) to mixtures containing 70 percent of gravel. In another series of tests two basic sand gradations were used and the filler content of the mixtures was varied. The third variable consisted of changing the asphalt content in the two test series outlined above. Specimens were prepared in quadruplicate for each condition of test in order to obtain good average data on which to base the comparison of the methods.

The test data obtained in this initial comparative series of tests and in other phases of this investigation are considered to be too voluminous for extensive presentation in this symposium; therefore, the findings in this comparative test series are discussed only in general terms,



General View Close-up

Figure 2. Adaptation of Marshall Stability Testing Head to Field CBR Testing Frame

and the detailed data are not presented herein.

Both the Marshall and the Hubbard-Field equipment were found to be sensitive to, and to detect by measurement, changes in gradation, variations in filler content, and changes in asphalt content. In either test the stability of the prepared samples increased with increasing asphalt content to some maximum value, after which the stability decreased. Both test methods indicated that a maximum stability was attained when the mixtures contained approximately 50 - 60 percent coarse aggregate in the particular blends used for these tests. Both test methods indicated that in sand-asphalt mixtures, where filler content was varied, the stability of the mixture increased with increasing filler content as indicated below. In the coarse-graded sand-asphalt mixtures, 15 percent filler produced maximum stability by both methods, and additional amounts of filler decreased the stability of the mixtures. In the fine-graded sand-asphalt mixtures, the stability by both test methods continued to increase with increasing filler content up to 20 percent, the maximum used in these tests. Results of the tests described above indicated that both test methods were sensitive in a comparable manner to changes in asphalt content and to changes in aggregate gradation and filler content.

Density determinations on specimens compacted as prescribed in the two test methods indicated that density of the compacted specimens increased with increments of asphalt cement to a maximum value, after which they decreased, For any given mix, however, the maximum unit weight, as determined by the high point of the curve, was greater in all cases for the Hubbard-Field than for the Marshall compaction procedure. A comparison with a very limited amount of field data available at that time indicated that compaction by the method used with the Marshall test more nearly duplicated densities obtained during normal construction than did compaction by the Hubbard-Field method.

In general, it was noted that the amount of asphalt required to produce maximum stability was roughly about 2 percent less in the Hubbard-Field than in the Marshall test. This difference is attributable to the greater densities obtained in samples compacted by the Hubbard-Field method. It was apparent that an optimum asphalt could be selected on the basis of

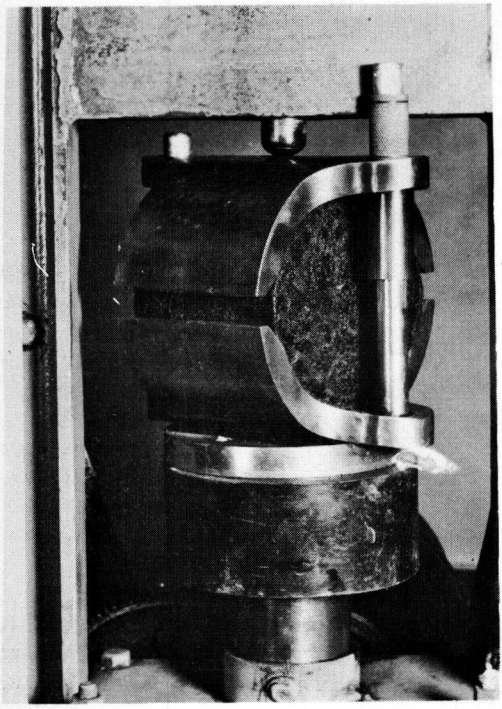


Figure 3. Marshall Specimen in Testing Position

stability by either test; the questionable factor being only the density to which the specimen need be compacted. The development of compaction technique was not considered pertinent to this phase of the study; however, subsequent laboratory work, described in a later paper, dealt very thoroughly with compaction procedures.

Comparative results on flow values were not possible in this investigation since the Hubbard-Field test did not include a comparable measurement. Flow values were measured, however, on the Marshall specimens, and it was noted that the flow value increased in a logical manner with increasing asphalt content. It was considered that when properly evaluated, the flow value would be a valuable measurement in the test procedure.

CONCLUSIONS

On the basis of the study briefly outlined above it was concluded that the Marshall equipment compared favorably with the recognized Hubbard-Field equipment as to measurement of stability, sensitivity to asphalt content, and reproduction of test results. Since the Marshall apparatus utilized equipment that could be readily incorporated into the CBR test apparatus and would be easily portable, it was decided to adopt the Marshall apparatus and to develop and perfect it for both design and control of bituminous pavements in the field.

LABORATORY STUDY OF ASPHALT PAVING MIXTURES

by W. K. BOYD*

INTRODUCTION

This paper summarizes the results of a portion of an extensive laboratory test program performed to study the basic factors that affect the quality of asphalt paving mixtures. Since only a very limited amount of work had been done previously with the Marshall equipment with respect to its selection for use, the new study also permitted an opportunity to develop and improve techniques and procedures for testing and for making a design analysis. In this laboratory program the factors that were studied to determine their effect on the quality of an asphalt paving mixture were: (a) amount of asphalt cement, (b) aggregate gradation, (c) aggregate type, (d) filler, and (e) penetratation grade of asphalt cement. Since the technique for sample preparation was the same for all tests, the factor of density produced by different compactive efforts was not a consideration. It is recognized that the density of a mix has an important bearing on its properties, as will be discussed in considerable detail in a later paper.

In order to evaluate and compare the quality of two or more asphalt mixtures it was necessary first to develop tools or yardsticks for measuring certain properties of each prepared test specimen. The selected Marshall test provided a means of measuring the strength and plasticity of a test specimen which could be expressed as numerical values. In addition, the specific gravity of each specimen could be determined and used in the computation of such properties as total weight, unit weight of aggregate only, from which such properties as aggregate and total weight, the percent voids in both the aggregate and total mix, and the percent of the total voids filled with asphalt. In all cases the properties named were determined for each specimen prepared and tested and they were used to compare and evaluate one specimen with respect to another. The effects of the different variables studied are presented in the following paragraphs.

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EFFECT OF ASPHALT CONTENT

In the entire laboratory study the variations produced by changes in asphalt content were the first variables to be compared. That is, for a given type and gradation of aggregate, a number of test specimens were prepared under exactly comparable conditions at each of several asphalt contents. The results of changes in asphalt content on the seven test properties chosen for study are presented for a typical example on Figure 1.

Stability - It can be seen on Figure 1 that the stability of a mixture increases with additional increments of asphalt to a maximum value and then decreases with further addition of asphalt. It is therefore possible to express, as numerical values, the comparative strength of any given mix for various asphalt contents. In the laboratory study the amount of asphalt required to produce maximum stability was the sole criterion for determining the asphalt content at which the other properties of asphalt mixtures would be compared. Later in this symposium it will be shown that other criteria for selecting optimum asphalt should also be considered. However, for the purpose of this laboratory investigation the properties at maximum stability provided a definite basis by which all mixtures could be evaluated and compared.

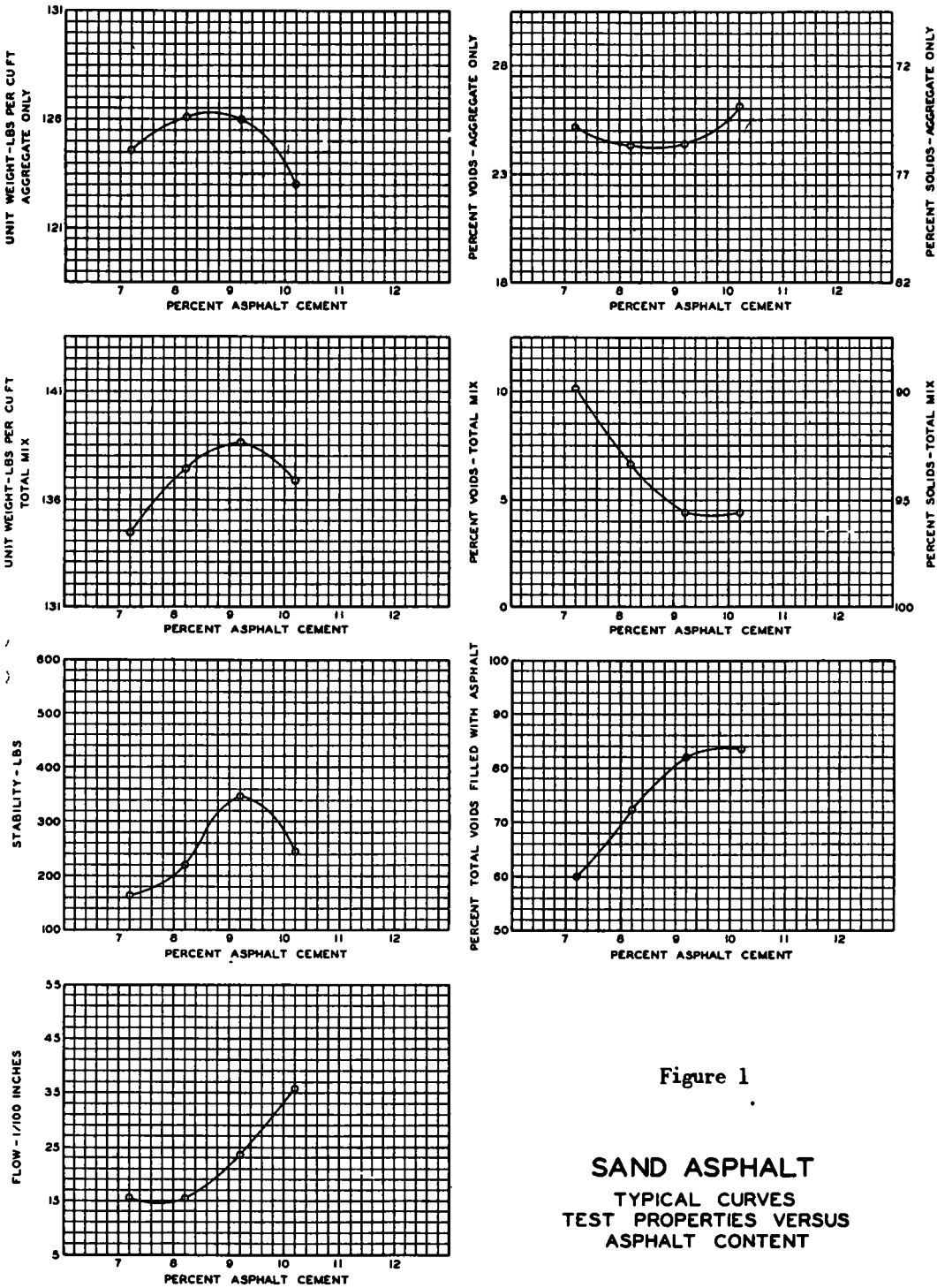


Figure 1

SAND ASPHALT

TYPICAL CURVES
TEST PROPERTIES VERSUS
ASPHALT CONTENT

Flow - The amount of plasticity inherent in a paving mixture for various percentages of asphalt may be measured with the Marshall equipment by the flow test. This test measures, in units of 1/100 of an inch, the amount of deformation required to produce failure of the test specimens. Referring to Figure 1, the flow value increases with the addition of asphalt. Referring to both the stability and flow curves, it can be seen by inspecting the stability curve that values of equal stability can be selected both below and above the optimum asphalt content. The flow curve shows that such values do not represent equally stable mixtures, since a mix with a lower flow value may be entirely stable, while the mix with the higher flow may be plastic and displace badly under traffic. For this reason stability must be associated with flow when comparison between two mixes is made. At optimum asphalt, however, the flow value of the mixes studied in the investigation varied only within a very narrow range. Since comparison of the other variables, aggregate gradation and aggregate type, will be at optimum, the flow value will not be a factor. It will be shown later in another paper that flow is considered an important measure of the quality of an asphalt mixture under traffic.

Density

Total weight - The total weight curve shown on Figure 1 indicates that the total weight of an asphalt mixture increases with additional increments of asphalt to a maximum value beyond which it decreases. The increase in total weight is accounted for by the fact that more asphalt provides greater lubrication which permits the aggregate to be seated together more compactly and each added increment (up to a certain limit) occupies a greater percent of the voids between aggregate particles, thus adding weight to the mass. For a given compactive effort a point is reached where the asphalt replaces aggregate in the mass and, being a lighter material, causes a reduction in the total weight of the specimen. For extremely coarse and open gradation, particularly asphaltic concretes, it was not possible to develop a typical asphalt versus total weight curve, because the voids between aggregate particles are excessively large.

Aggregate weight - With the total weight of the specimen determined by test and knowing the percent of asphalt that has been added, the unit weight of the aggregate portion may be found by computation. A typical curve in which the aggregate weight is compared with variation in asphalt is shown on Figure 1. In all cases the amount of asphalt required to produce maximum aggregate density was less than that required for maximum total density. By examining the aggregate weight curve it can be seen that there is a small range of asphalt contents over which the aggregate structure remains approximately constant. Any increase in the amount of asphalt in this range is utilized in filling additional voids and adds weight to the total mix. Further additions of asphalt above this range separate the particles and cause a loss in aggregate weight, but because this loss may be offset by an increment of asphalt which fills a greater amount of the remaining voids, the total weight may increase. Therefore, in all cases the curves based on aggregate weight reached a maximum value at a lesser amount of asphalt than did the curves based on total weight.

Percent voids total mix - The portion of the total volume of a specimen occupied by aggregate and asphalt comprises the solid fraction and may be termed the percent by volume of solids in the paving mixture. The complement of percent solids is percent voids total mix. On Figure 1 a curve for this test property is plotted with a scale showing the percent voids total mix on one side and the percent solids on the other side. Referring to these curves, it is apparent that as asphalt is added the voids in the aggregate fraction become less until a point is reached where the mass contains a minimum of voids, and any further increase in asphalt spreads the aggregate particles apart leaving the percent voids virtually unchanged. Therefore the point on the curve where the rate of curvature changes indicates the percent asphalt required to secure the maximum practical density.

Percent voids aggregate only - Aggregate weight of a test specimen may be converted to volume by calculation and expressed as a percent of the total volume of the mixture. The ratio of the volume occupied by aggregate to the total volume of the mass is termed percent solids aggregate only. The complement of this value is termed percent voids aggregate only. The amount of asphalt that produces the greatest aggregate weight is the same as will produce the least percent voids in the aggregate.

Percent voids filled with asphalt - The ratio of the volume of asphalt in a mix to the aggregate voids may be expressed as the asphalt-void ratio. For example, if the aggregate occupies 80 percent of the volume of a test specimen, the voids available for asphalt are 20 percent. If asphalt fills 15 percent of the aggregate voids, then 15/20 or 75 percent of the available voids are filled.

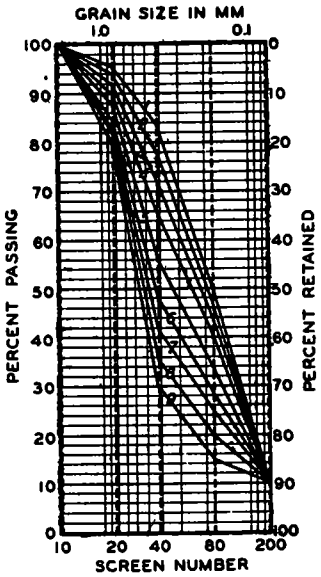
From Figure 1 it can be seen that a higher percentage of voids is filled with each additional increment of asphalt. Since it has no peak value as is the case with some of the other properties, and bacause the values at optimum asphalt for satisfactory graded mixes vary only within a narrow range of values, the property is of little importance in this paper. However, the property will be shown to have considerable importance in the papers to be given later.

Importance of asphalt content - It is evident from the discussion just presented that the test properties are affected a great deal by the amount of asphalt that is included in an aggregate mixture. If a very small amount is used, the resulting mix is brittle, has low strength or stability, the weight is comparatively low, and the voids excessively large. If too much asphalt is used, the resulting mix is too plastic, has little strength and also may be considered unsatisfactory. It is considered that asphalt content is the most critical variable in a paving mixture. In the following paragraphs asphalt mixtures will be compared in which other factors are variables. In all cases the values shown will reflect the test properties of an asphalt mixture at its optimum asphalt content as determined by the stability test. The data presented are summaries only. In all cases a series of test specimens was prepared in which asphalt content was the variable and from which optimum asphalt was selected.

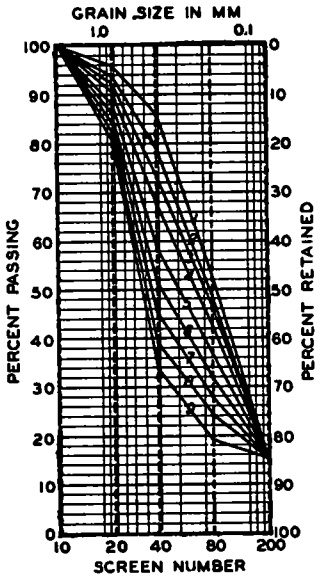
EFFECT OF AGGREGATE GRADATION

Sand Asphalt - In order to investigate the effect of aggregate gradation on sand asphalt, nine blends of fine aggregate covering a wide range of gradation were prepared. The blends are designated gradations 1 through 9, with gradation 1 the finest and gradation 9 the coarsest mix. These blends provided fine aggregates, all of which passed the No. 10 sieve, varying from a coarse to a very fine gradation. In general, the principal difference between the nine gradations occurred in the amount of material passing the No. 40 sieve with minor differences in the amount of material passing the No. 80 sieve. Sufficient limestone dust was added to each of these nine basic blended materials to give mixtures having a total of 10, 15, and 20 percent of filler. In addition, limestone dust was added to gradations 1, 3, 5, 7, and 9 to give mixtures having a total of 25 percent filler. The gradations of these blends are shown on Figure 2.

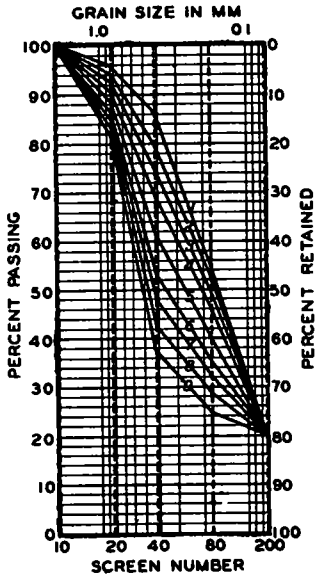
On Figure 3 the results of variation in fine aggregate gradations are shown by curves for all test properties. In all cases the curves drawn through the plotted points represent the changes that result when the gradation of the fine aggregate is varied with the quantity of filler held constant. The curves representing flow (Figure 3) indicate no significant trend and for the purpose of this analysis it is assumed that they are all satisfactory. All curves representing stability show a progressive increase in numerical value from gradations 1 through 7 with a tendency for lower values on gradations 8 and 9. Unit weights (both total and aggregate) show a progressive increase in numerical value from gradations 1 through 8 with a tendency for lower values for gradation 9. The percent voids (total mix and aggregate) generally decreased progressively from gradations 1 through 7



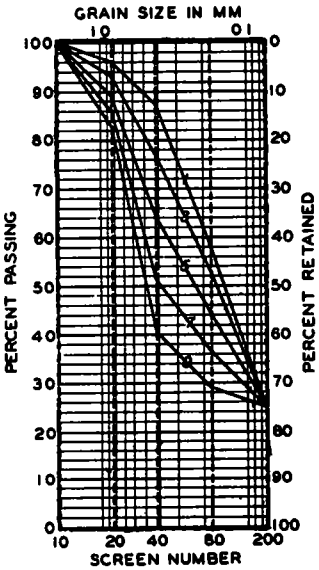
10% FILLER
FIGURE 1



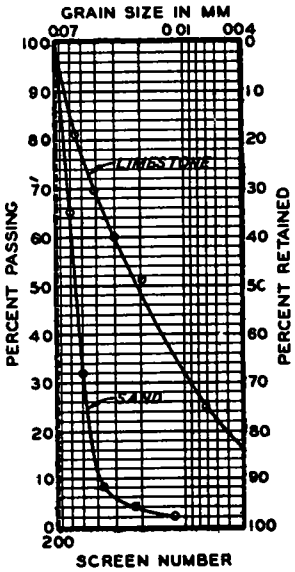
15% FILLER
FIGURE 2



20% FILLER
FIGURE 3



25% FILLER
FIGURE 4



SAND AND LIMESTONE FILLER
FIGURE 5

AGGREGATE GRADING CHARTS

SAND ASPHALT

Figure 2

with a tendency to remain constant or increase for gradations 8 and 9. The curves representing asphalt-void ratios indicate a slight trend to fill more of the voids with asphalt progressively from gradations 1 through 9. The data indicate that increasing the amount of fine aggregate retained on a No. 40 screen to as much as

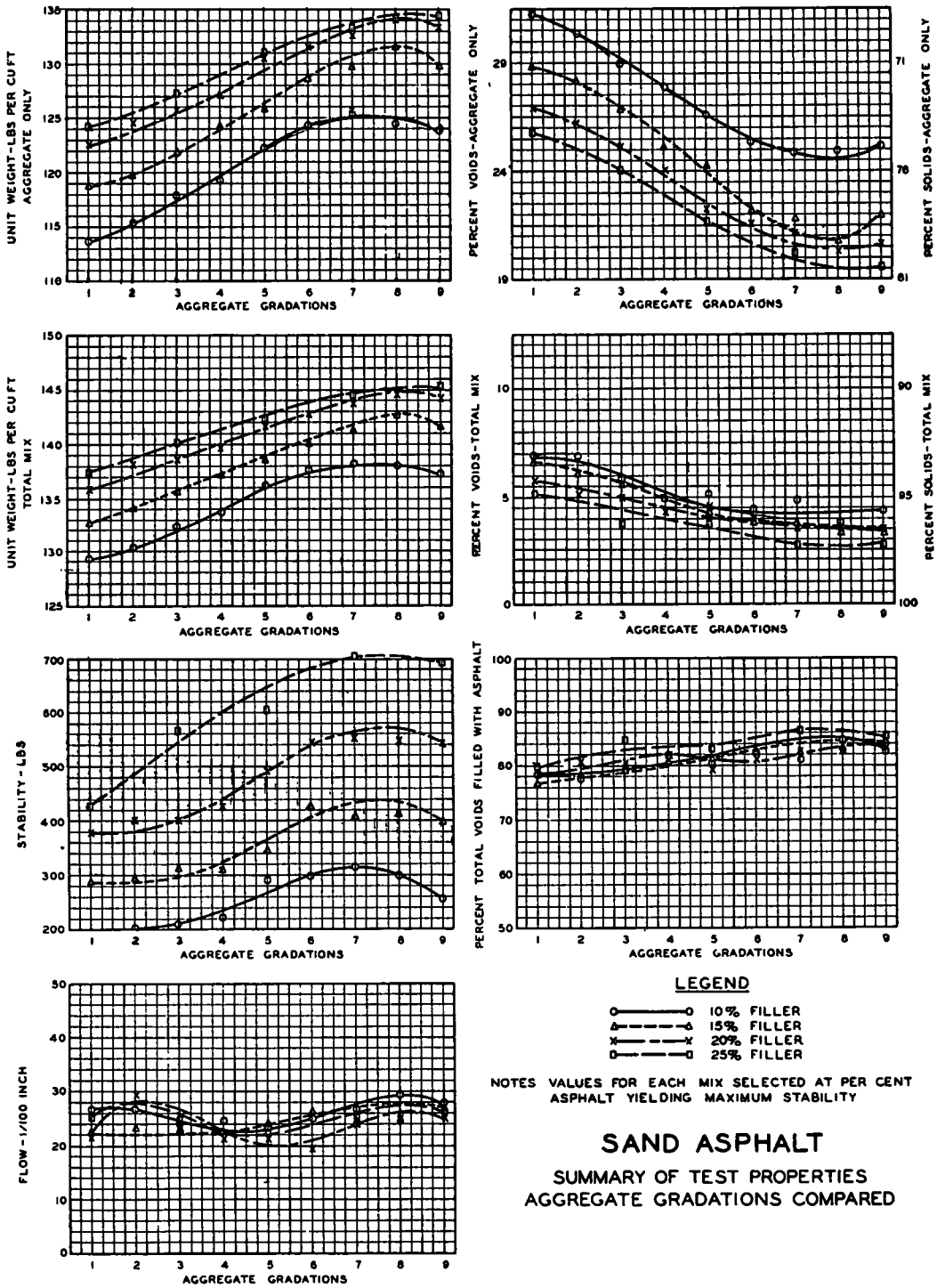
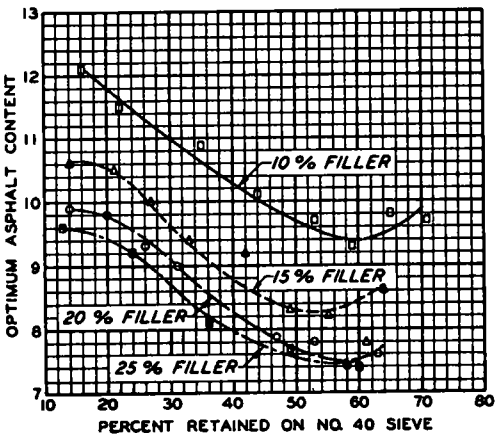

UNIT WEIGHT-LBS PER CU FT AGGREGATE ONLY
AGGREGATE GRADATIONS
PERCENT VOIDS-AGGREGATE ONLY
PERCENT SOLIDS-AGGREGATE ONLY
UNIT WEIGHT-LBS PER CU FT TOTAL MIX
PERCENT VOIDS-TOTAL MIX
PERCENT SOLIDS-TOTAL MIX
STABILITY-LBS
PERCENT TOTAL VOIDS FILLED WITH ASPHALT
FLOW-1/100 INCH
LEGEND
10% FILLER
15% FILLER
20% FILLER
25% FILLER
NOTES VALUES FOR EACH MIX SELECTED AT PER CENT ASPHALT YIELDING MAXIMUM STABILITY
SAND ASPHALT
SUMMARY OF TEST PROPERTIES
AGGREGATE GRADATIONS COMPARED

Figure 3.



SAND ASPHALT

SUMMARY OF OPTIMUM ASPHALT CONTENTS

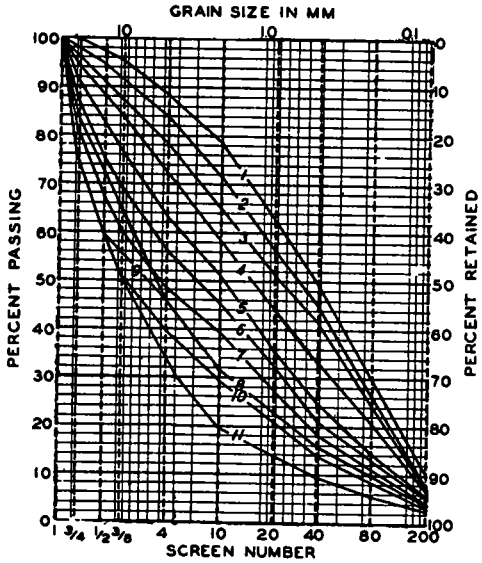
Figure 4.

about 55 to 65 percent of the total aggregate improved a sand asphalt mixture, the exact amount being contingent on the amount of filler present in the mixture. It is not inferred that sand asphalt mixtures containing finer gradations are unsatisfactory, as no limits for the test properties have been established.

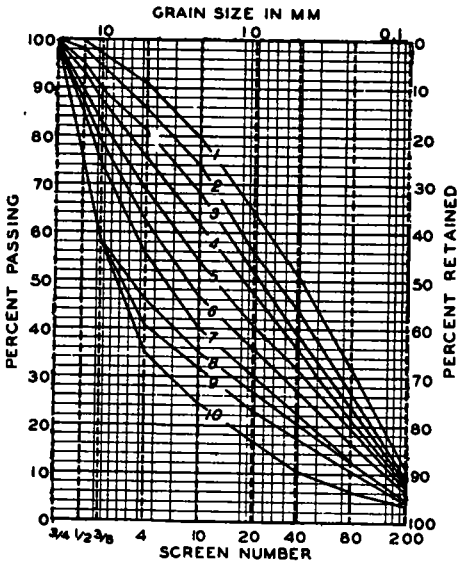
Effect on optimum asphalt - On Figure 4 the amount of asphalt determined as optimum is shown by a curve for each variation in aggregate gradation and for each of four filler contents. It can be seen that the amount of asphalt required for optimum was reduced with incremental increases in the amounts of coarse sand until about 50 to 60 percent of the total aggregate is retained on the No. 40 screen. A further increase in coarse sand again increased the optimum asphalt content. This trend was duplicated by the curves for percent voids aggregates only shown on Figure 3, and suggests that the amount of asphalt required is dependent on the available voids. If the four curves on Figure 4 are compared, it is seen that they are in the order of filler content. That is, a given aggregate gradation with 15 percent filler required less asphalt for optimum than the same gradation with 10 percent filler, still less for 20, and least with 25 percent filler. It is apparent that both asphalt and filler are void filling materials and will supplement each other (within limits) in a paving mixture.

Asphaltic concrete - In order to investigate the effect of aggregate gradations on asphalt concrete mixtures, ten blends each were prepared for the 3/4 and ½-in. maximum aggregate size and eleven blends for the 1-in. size. Blends have been designated gradation 1 through 10 (or 11), blend 1 being the finest. Gradation curves for the blends are shown on Figure 5. Test series conforming to the gradation curves shown were prepared using uncrushed gravel and slag as the coarse aggregate. Also, test series were made conforming to the gradation curves for 3/4 in. maximum size using crushed limestone and crushed gravel, in addition to the series for uncrushed gravel and slag. In all cases the fine aggregate fraction consisted of a blend of two local sands. Limestone dust was used as the filler. In general, the principal variation in the gradation curves occurred in the coarse aggregate fraction. If the gradations of the fine aggregates were replotted on the basis of 100 percent passing a No. 10 sieve, they would be reasonably comparable.

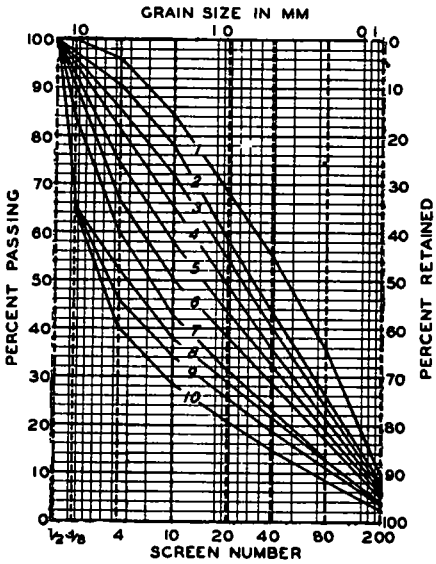
Uncrushed gravel - Summary test results on specimens prepared with uncrushed gravel in which the percent of coarse aggregate varied from about 15 to 80 percent of the total aggregate and for three (½-, 3/4-, and 1-in.) maximum size materials are shown on Figure 6. With each added increment of coarse aggregate the stability and all density properties were improved until the mixture was composed of approximately 60 to 70 percent coarse aggregate. There was a rather indefinite trend in the percent voids filled with asphalt to increase until more than 70 percent coarse aggregate was included in the mixture; beyond 70 percent the value dropped markedly. It is clear that in these tests stability was dependent on density, which means that (for any given aggregate type) the gradation which permits the least aggregate voids, or the highest density, will usually possess the highest structural strength. From visual inspection of the test specimens, it was observed that those containing more than 70 percent coarse aggregate would not



1 INCH MAXIMUM
FIGURE 1



3/4 INCH MAXIMUM
FIGURE 2



1/2 INCH MAXIMUM
FIGURE 3

NOTE SCREEN NUMBERS REFER TO U S STANDARD SIEVE

AGGREGATE GRADING CHARTS
ASPHALTIC CONCRETE

Figure 5

produce a desirable wearing course paving mixture because of their open texture. The curves for ½, 3/4, and 1 in. maximum size aggregate also show that there was an improvement in both stability and density (slight) as the maximum size increased. Therefore, in general, both for economy (less crushing, screening, etc.) and greater strength, the largest maximum size aggregate consistent with pavement

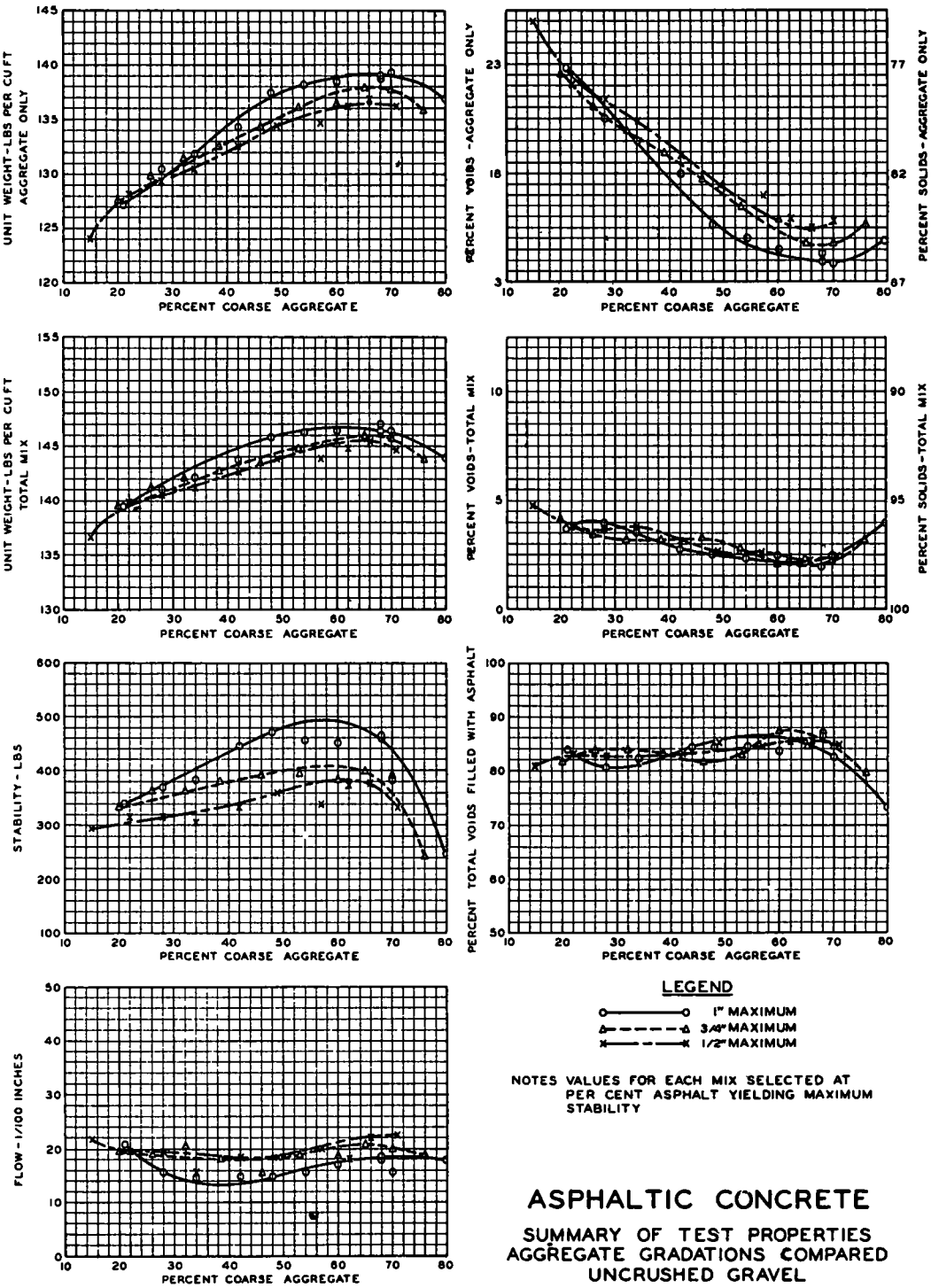
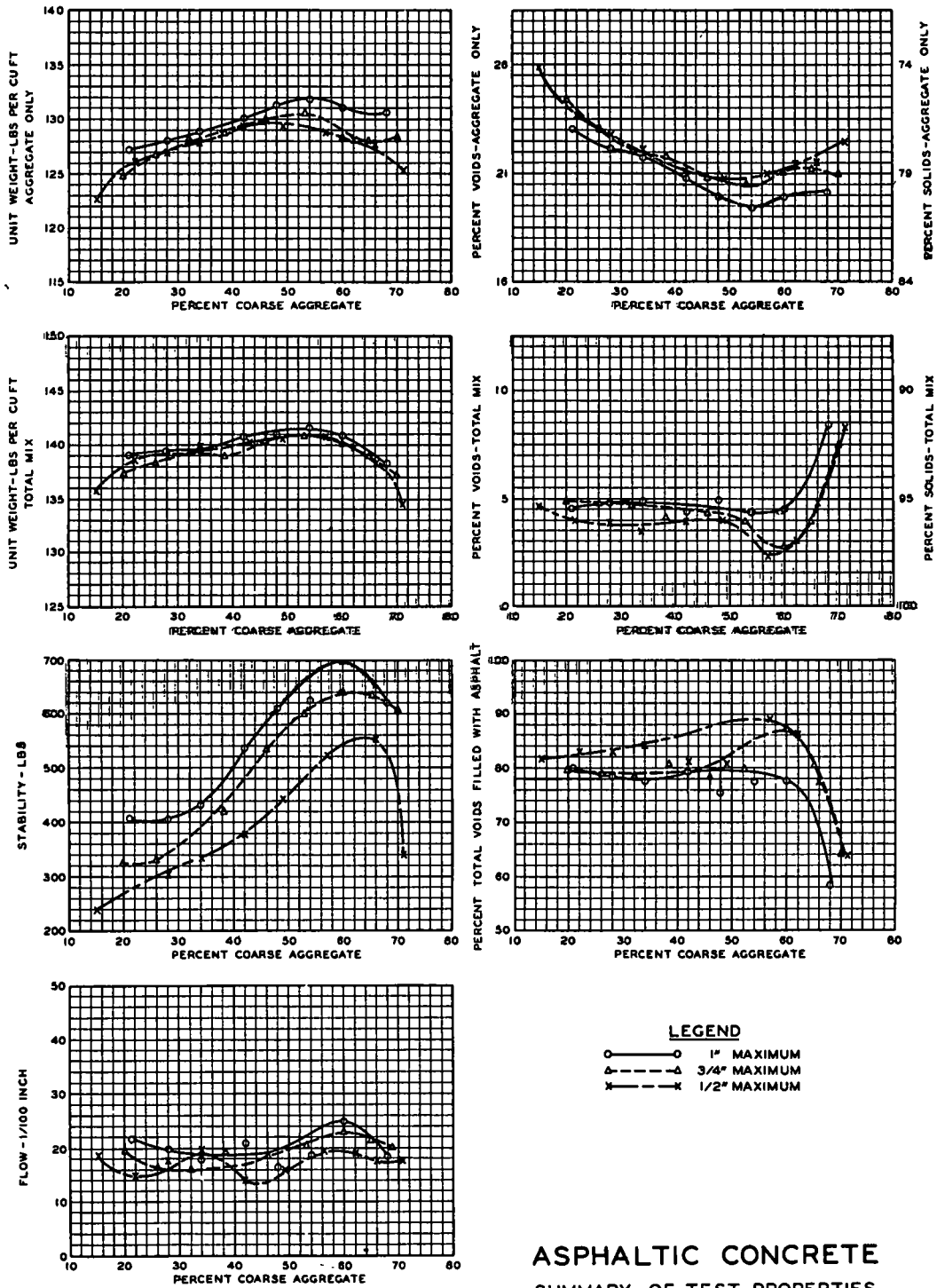

UNIT WEIGHT-LBS PER CU FT AGGREGATE ONLY
PERCENT COARSE AGGREGATE
PERCENT VOIDS-AGGREGATE ONLY
PERCENT SOLIDS-AGGREGATE ONLY
UNIT WEIGHT-LBS PER CU FT TOTAL MIX
PERCENT VOIDS-TOTAL MIX
PERCENT SOLIDS-TOTAL MIX
STABILITY-LBS
PERCENT TOTAL VOIDS FILLED WITH ASPHALT
FLOW-1/100 INCHES
LEGEND
1" MAXIMUM
3/4" MAXIMUM
1/2" MAXIMUM
NOTES VALUES FOR EACH MIX SELECTED AT PER CENT ASPHALT YIELDING MAXIMUM STABILITY
ASPHALTIC CONCRETE
SUMMARY OF TEST PROPERTIES
AGGREGATE GRADATIONS COMPARED
UNCRUSHED GRAVEL

Figure 6



NOTES VALUES FOR EACH MIX SELECTED AT PER CENT ASPHALT YIELDING MAXIMUM STABILITY

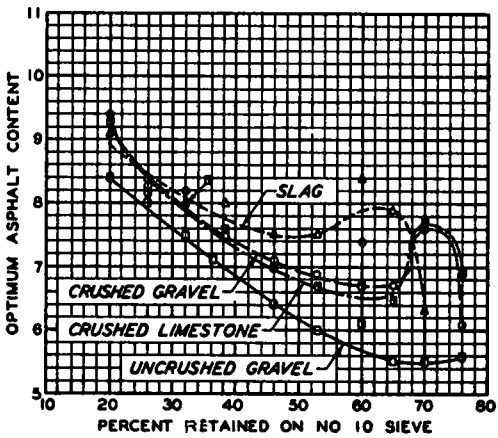
ASPHALTIC CONCRETE

SUMMARY OF TEST PROPERTIES
AGGREGATE GRADATIONS COMPARED
SLAG

Figure 7

thickness and construction should be used.

Slag - In general, the statements concerning uncrushed gravel, are true for mixtures containing slag as the coarse aggregate fraction, as shown on Figure 7. However, maximum values for stability and density were secured at about 60 percent coarse aggregate for the slag rather than the slightly higher range indicated for uncrushed gravel. This was due to the facts that slag was a much lighter material and that all percentages were based on weight, the volume of slag being greater for equal weights of the two materials. The specific gravity of a material should therefore be considered when any statement concerning maximum amounts of coarse aggregate is made.



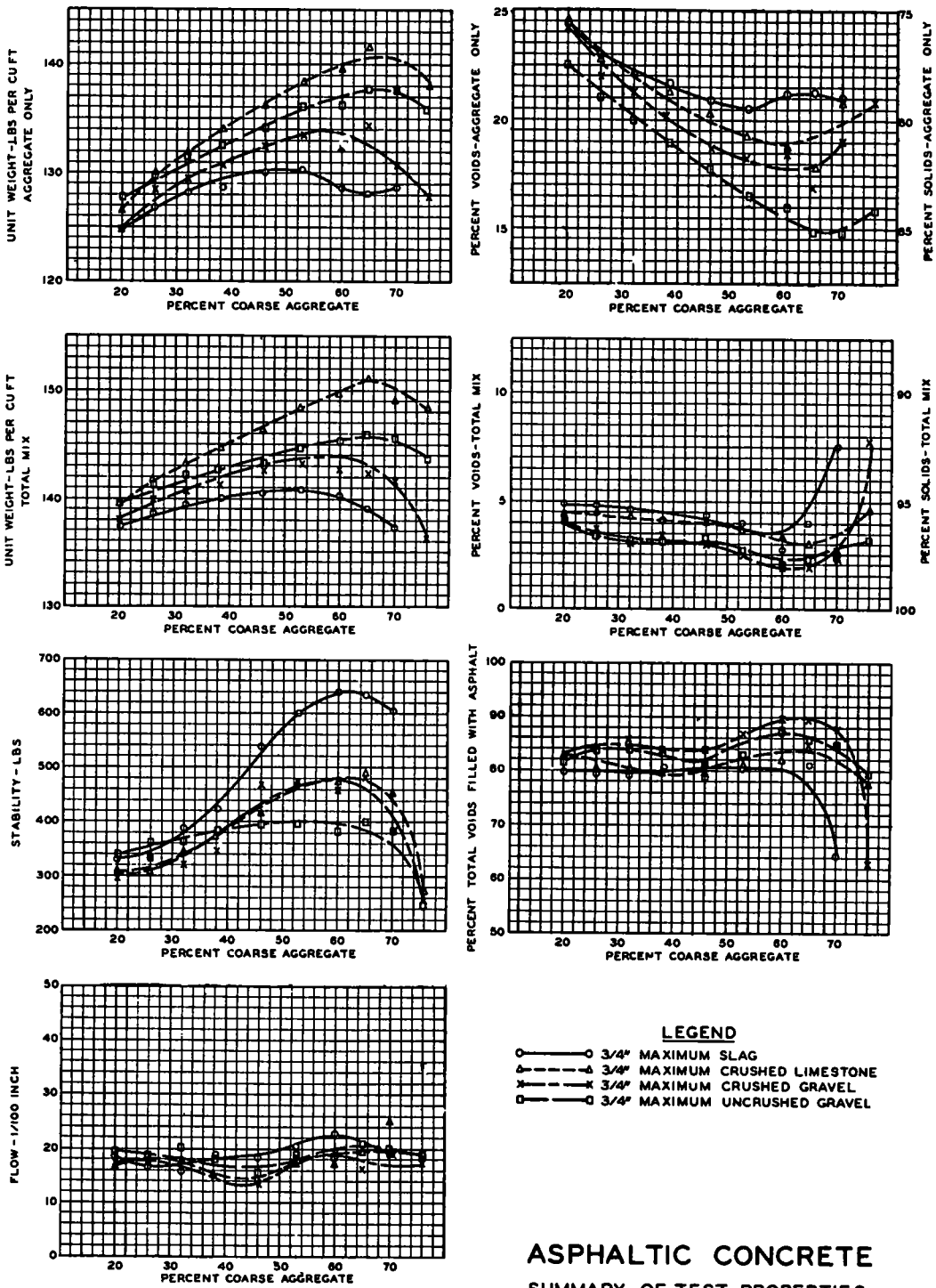
ASPHALTIC CONCRETE
SUMMARY OF OPTIMUM ASPHALT CONTENTS
3/4 INCH MAXIMUM SIZE

Figure 8

Effect on optimum asphalt - On Figure 8 is shown the amount of asphalt indicated as optimum for different aggregate gradations and aggregate types. All of the curves have the same general characteristics and show that less asphalt was required with each additional increment of coarse aggregate added to the paving mixture until the total amount of coarse aggregate exceeded about 55 percent for slag and about 65 percent for crushed limestone and gravel. Below these limits the trend for less asphalt corresponds directly with the reduction in the aggregate voids (Figure 9). Since at optimum about the same percent of the aggregate voids was always filled with asphalt, the amount of asphalt at optimum must be reduced with each incremental increase in coarse aggregate. It may be noted that there was a point (65 percent for slag and 70 percent for crushed limestone and gravel) where the ratio between aggregate voids and percent asphalt in the mix was not constant. Apparently the voids became excessively large so that they were not filled to the same extent as were smaller voids, and the amount required for optimum asphalt was sharply reduced. As previously discussed, all test specimens in the laboratory study were compacted at one compaction effort. It will be shown in later papers that this compaction effort was low. A greater compaction effort will produce higher densities, reduce aggregate voids, and any given mixture will consequently require less asphalt for the optimum value than is shown by these data. Therefore, the data should be compared only from a qualitative standpoint.

EFFECT OF AGGREGATE TYPE

A summary of test properties for four types of coarse aggregate 3/4 in. maximum size, is presented on Figure 9. It should be noted that the various density curves show the same trends with change in amount of coarse aggregate and, with respect to each other, reflect the difference in their specific gravities. Slag had the lowest apparent specific gravity and the greatest amount of voids in the aggregate of the four types compared. It also had the highest stability. Crushed limestone was the heaviest material; yet the aggregate voids were high, being exceeded only by the slag. It is apparent that comparisons of types of aggregate on a weight basis for specification purposes are not necessarily valid while limiting values for stability, flow, percent voids aggregate only, percent voids total mix, and percent voids filled with asphalt may be established which apply to all aggregates. However, as has been discussed in previous paragraphs, a weight basis is of



NOTES VALUES FOR EACH MIX SELECTED AT PER CENT ASPHALT YIELDING MAXIMUM STABILITY

ASPHALTIC CONCRETE

SUMMARY OF TEST PROPERTIES TYPES OF COARSE AGGREGATE COMPARED

Figure 9

considerable importance in the design analysis and construction control of a specific asphalt paving mixture. With regard to stability (see Figure 9), the stability curves were grouped closely together when the coarse aggregate was less than about 40 percent of the total mix (this assumes that about 34 percent slag by weight was equivalent in volume to 40 percent of the other materials). For mixtures containing more than 40 percent coarse aggregate, the curves begin to spread with the stability for crushed materials higher than for uncrushed material, slag being the highest of all materials tested. Apparently when the coarse aggregate was less than 40 percent of the total, it was effective principally in reducing the voids and increasing the density with each particle floating in the matrix independently of the others. When present in excess of about 40 percent, the quality of the material became effective by particle to particle contact, as reflected by higher stability values for crushed materials and slag when compared to uncrushed gravel.

FILLER

Effect of amount of filler - From Figure 3 it can be seen that 15, 20, and 25 percent filler in the nine basic gradations show progressive improvements in the properties over those secured for respective gradations with 10 percent filler. The data on Figure 4 show that as the amount of filler is increased, optimum asphalt content is reduced. The function of filler in a mix is readily apparent from a review of these two figures. Incremental increases in filler up to about a maximum of 20 percent increased the stability, increased the total and aggregate weight, reduced both the aggregate and total voids, and required less asphalt. It is believed that the amount of filler selected for a particular design should include the following considerations: (a) the amount required to produce satisfactory stability, (b) the amount required to produce satisfactory void values, (c) the gradation of the aggregate, and (d) relative cost of asphalt cement, aggregate, and filler. There are other considerations which may modify the maximum amount of filler that is desirable in a mix, such as the flexibility of a mix or the tendency to check and crack during construction rolling; however, these factors were not determined by the laboratory study. In Appendix B of Waterways Experiment Station Technical Memorandum No. 3-254*(1)* a method of analysis is presented which indicated that, based on laboratory tests only, the maximum amount of filler for any of the mixes shown was about 20 percent.

Effect of type of filler - The test results obtained by substituting 10, 15, 20, and 25 percent sand passing a No. 200 mesh sieve for equal quantities of limestone filler are shown for a typical case on Figure 10. It is apparent that equal amounts of sand filler do not improve the desirable properties of a mixture (increased stability and density, decreased voids) to the same extent as limestone filler. Increasing the quantity of limestone filler (up to a certain point) increased stability and density and decreased the voids; however, this was not necessarily true for sand filler, and it can be seen in some cases that additional increments of sand filler were actually detrimental. A gradation curve for each type of filler is shown on Figure 2. The sand filler was composed largely of material between 0.07 and 0.03 mm in size, whereas the limestone filler was well graded with approximately 17 percent finer than 0.004 mm. If the properties of an asphalt mixture are to be improved by adding filler, it is essential that the filler be well graded and that it include material finer than 0.01 mm in size. Otherwise, filler may become a void-producing rather than a void-filling material.

EFFECT OF PENETRATION GRADE OF ASPHALT

Tests were conducted to determine the effect of penetration grade of asphalt cement on the test properties. Four grades of asphalt cement having penetrations of 53, 80, 117, and 135 were used. Results of these tests are summarized on Figure 11. All tests were conducted at optimum asphalt content. It can be seen that the

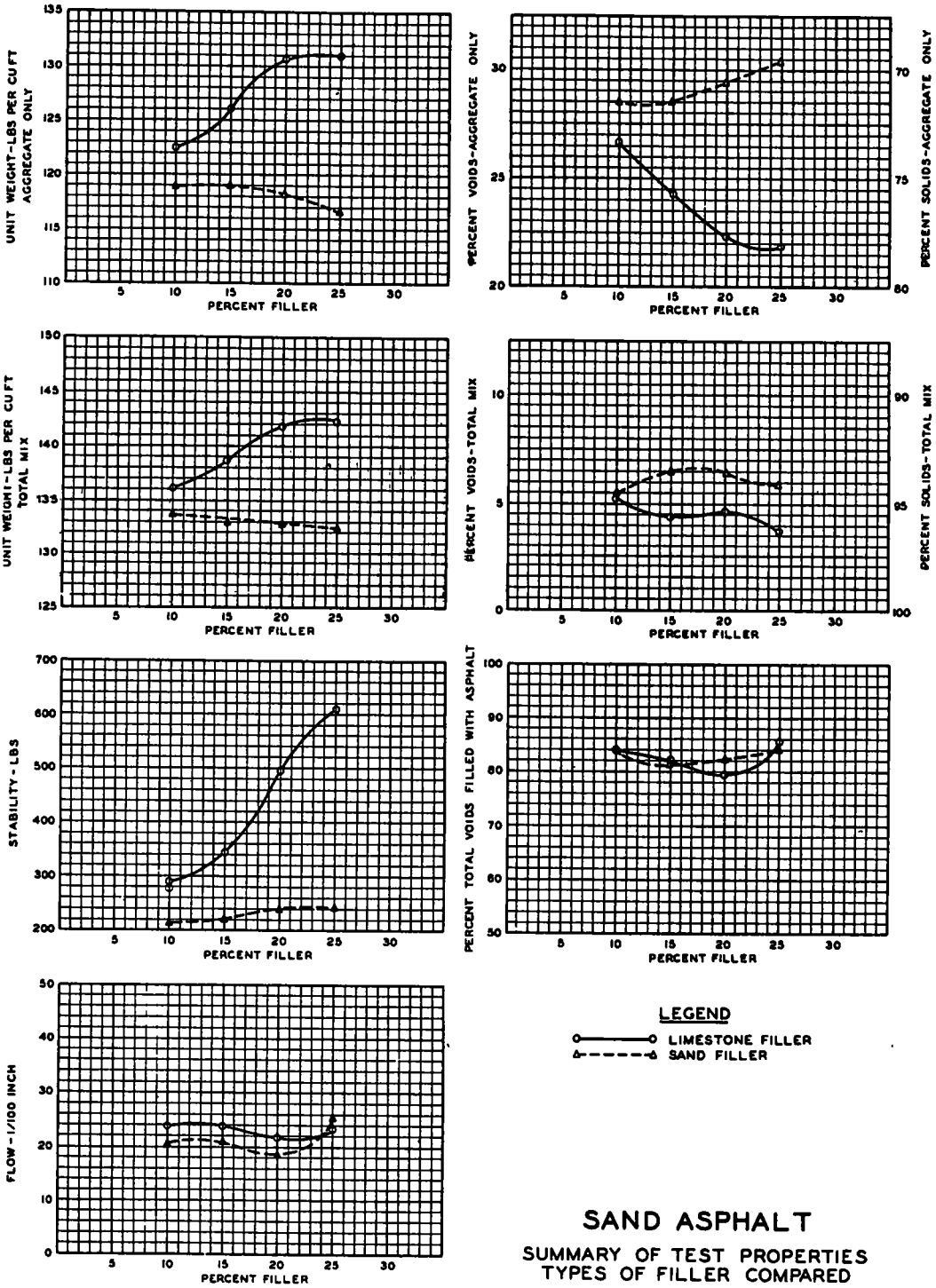
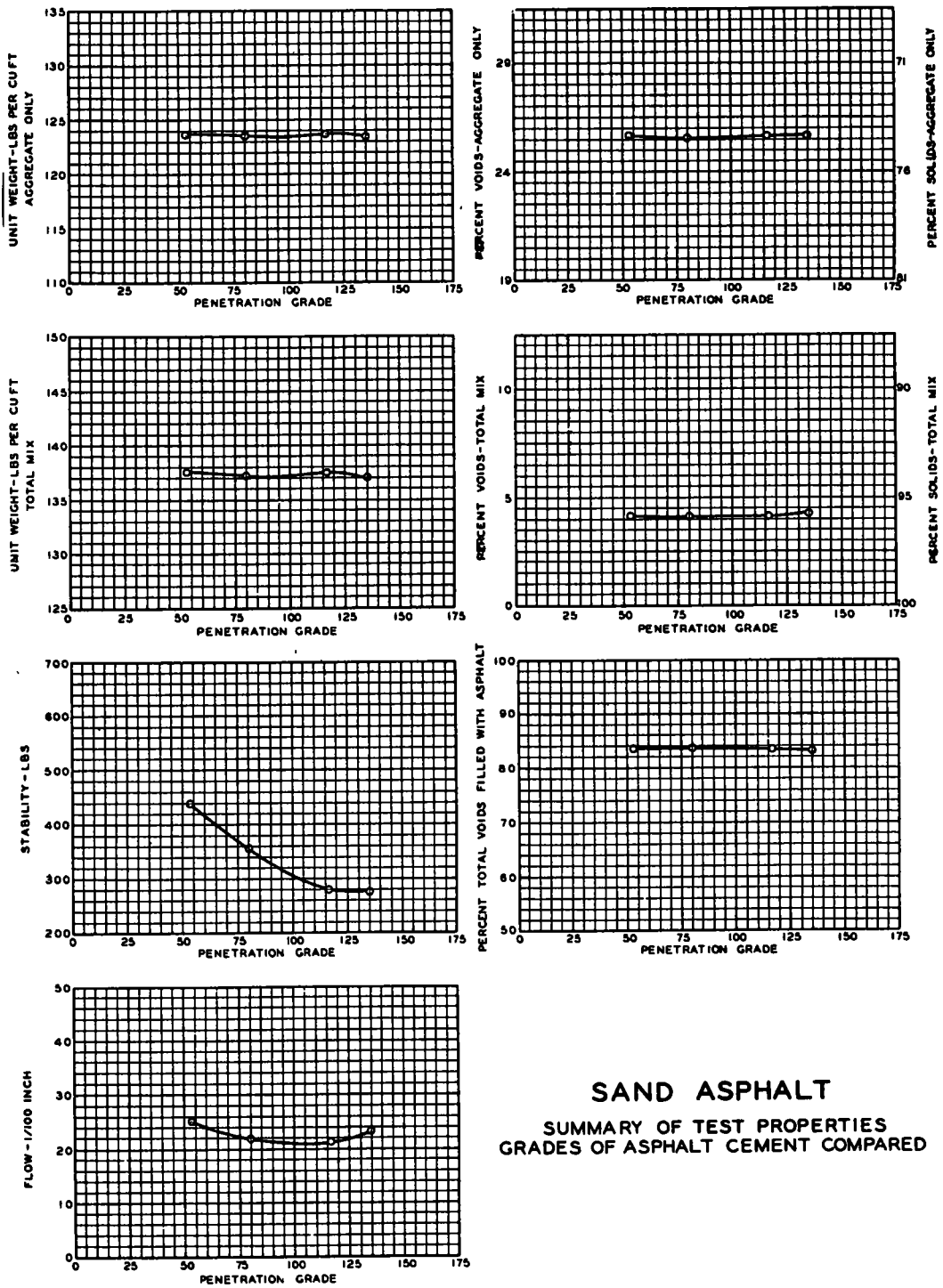

UNIT WEIGHT-LBS PER CU FT AGGREGATE ONLY
PERCENT FILLER
PERCENT VOIDS-AGGREGATE ONLY
PERCENT SOLIDS-AGGREGATE ONLY
UNIT WEIGHT-LBS PER CU FT TOTAL MIX
PERCENT VOIDS-TOTAL MIX
PERCENT SOLIDS-TOTAL MIX
STABILITY-LBS
PERCENT TOTAL VOIDS FILLED WITH ASPHALT
FLOW-1/100 INCH
LEGEND
LIMESTONE FILLER
SAND FILLER
SAND ASPHALT
SUMMARY OF TEST PROPERTIES
TYPES OF FILLER COMPARED

Figure 10



SAND ASPHALT

SUMMARY OF TEST PROPERTIES
GRADES OF ASPHALT CEMENT COMPARED

Figure 11

use of different grades of asphalt cement had little or no effect on any property except stability. Increased Marshall stability can be secured by using a lower penetration asphalt.

CONCLUSIONS

The conclusions given in the following paragraphs are believed warranted on the basis of the information and data presented in this paper. A more complete discussion and analyses of the laboratory study, together with additional conclusions, may be found in Appendix B of Waterways Experiment Station TM 3-254(*1*).

The Marshall stability test is an empirical test whereby the relative strength of basically different asphaltic mixtures can be compared.

The flow test, an integral part of the stability test, measures the relative plasticity of asphaltic mixtures.

Incremental increases of asphalt cement to asphalt mixtures will increase the amount of flow; therefore, mixtures of equal stability which contain unequal amounts of asphalt cement may be compared on the basis of flow.

Incremental increases of asphalt to a given aggregate mixture will produce an increase in the total weight to some maximum value, after which further increases of asphalt will cause the total weight of the mixture to decrease.

In general, an increase in the density produces an increase in the stability of a mixture.

Density, expressed as unit weight, cannot be used in a relative comparison of different types of aggregate mixtures; however, it is considered to be of practical value in the design analysis and control of a specific asphalt mixture.

The amount of asphalt required to produce maximum density of the aggregate only is less than that required to produce maximum total weight.

The percent voids total mix is dependent on (a) asphalt content, (b) aggregate gradation, (c) filler content, (d) compactive effort, and (e) aggregate type.

Where the asphalt content is the only variable, the selection of a percent asphalt required to produce maximum aggregate density may be made on the basis of the amount of aggregate voids.

The quantity of asphalt cement required for optimum asphalt content decreases as the aggregate voids decrease.

Well graded fine and coarse aggregate mixtures prepared at optimum asphalt and with a prescribed compactive effort will have an asphalt-void ratio that varies within very narrow limits.

Very open or harsh-graded aggregates, when used in asphaltic mixtures, give an asphalt-void ratio at optimum asphalt content considerably less than that of well-graded aggregates.

The addition of coarse sand (No. 10-40) up to about 60 percent of the total aggregate improved the test properties of sand asphalt mixtures for the materials tested.

While the addition of coarse sand up to about 60 percent improved the test properties, the definite limit of coarse sand depends on the gradation of the fine fractions of the aggregate (No. 40-200) and the amount and type of filler.

The measured properties of an asphaltic concrete mixture were improved by the addition of coarse aggregate to a maximum of between 60 and 70 percent of the total aggregate.

An increase in the maximum size of aggregate (from ½ to 1 in.) improved the test properties of the mixture.

The test properties of a mixture were not changed by the type of aggregate (slag, crushed gravel, uncrushed gravel, or crushed limestone) when present in the mixture in amounts less than 40 percent.

When the amount of coarse aggregate exceeded 40 percent, the type of aggregate became effective; slag produced the highest stability, crushed gravel and crushed limestone were about equal and produced the next highest stability, and uncrushed gravel produced the lowest stability.

The addition of filler to an aggregate mixture increased the stability, total and aggregate weight, and reduced the aggregate and total voids and the amount of asphalt required for optimum.

The maximum amount of filler to be

added to an asphalt mixture was not determined and is dependent on several factors: (a) amount necessary to produce a satisfactory stability, (b) amount necessary to meet specified void requirements, (c) aggregate gradation requirements, (d) relative cost of asphalt cement, sand, and filler, and (e) other possible factors not considered in this study, such as durability and flexibility.

The addition of a well-graded filler to a sand asphalt mixture improved the measured test properties.

The addition of a poorly-graded sand filler did not improve the measured test properties of a sand asphalt mixture.

The use of different penetration grades (between 50 and 135) of asphalt cement had no effect on any measured property except stability which was increased with each decrease in the penetration grade of asphalt.

ASPHALT STABILITY TEST SECTION

by C. R. FOSTER*

INTRODUCTION

In the studies discussed by Messrs. Griffith and Boyd, the significant properties of an asphalt paving mixture were discussed and it was shown that each blend of aggregates has an optimum asphalt content. Procedures were developed for determining the optimum asphalt content and for comparing various mixes which appeared reasonable on the basis of the laboratory tests. It was recognized, however, that before laboratory tests can be finally evaluated and before the results obtained from such tests can serve as design criteria, the tests must be correlated with actual behavior of pavements under traffic. Accordingly, a test section containing a wide variety of pavements was constructed and subjected to traffic with simulated airplane wheel loads. Specimens of the pavements were obtained and tested, and observations of the behavior of the pavements were made at regular intervals. This paper described briefly the test section, the application of traffic, and the analysis of the data to determine design criteria. More complete details are given in Appendixes C and D to Waterways Experiment Station TM 3-254 *(1)*.

Seven objectives for the test section were embraced in the formal statement. Without repeating the wording of that statement, it will suffice to say that the essential objectives were to obtain test data necessary for the development of design criteria for mixture and thickness requirements for pavements on bases of various qualities. In this paper the term "mixture" is used when there is no reference to a component part of a flexible pavement cross section; the term "pavement" is used where a reference is made to the component part of a flexible pavement section. Wheel loads of 15,000, 37,000, and 60,000 pounds were specified in the objectives.

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TESTS

Test Section

Figure 1 shows a plan of the test section and Figure 2 shows a view of the completed project. The plan for the test section and the subsequent testing program were prepared by the Waterways Experiment Station in conjunction with personnel of the Office, Chief of Engineers and the Engineer Board (now the Engineer Research and Development Laboratories). As shown on Figure 1, the test section consisted of two parallel straight tracks, each 850 ft. long by 60 ft. wide, joined at each end with turnarounds 175 ft. by 175 ft. The straight tracks were divided longitudinally into three lanes, designated a, b, and c, for the three wheel loads.

Subgrade - The test section was constructed in an upland area where the natural soil was a loessial clayey silt with an average liquid limit of 47 percent and plasticity index of 23 percent. The soil was compacted to 93 percent modified AASHO density and the CBR (unsoaked) was evaluated at 30 percent. Since the project was a study of pavements and not a study of subgrades, it was desired that no failures develop in the subgrade. In order to insure that no failures would develop in the subgrade, the total thickness of base and pavement was set at 9, 13, and 16 in. for the 15,000, 37,000, and 60,000-lb. wheel loads, which was about 20 percent greater than called for by the Corps of Engineers' design curves current in 1945. No failures occurred in

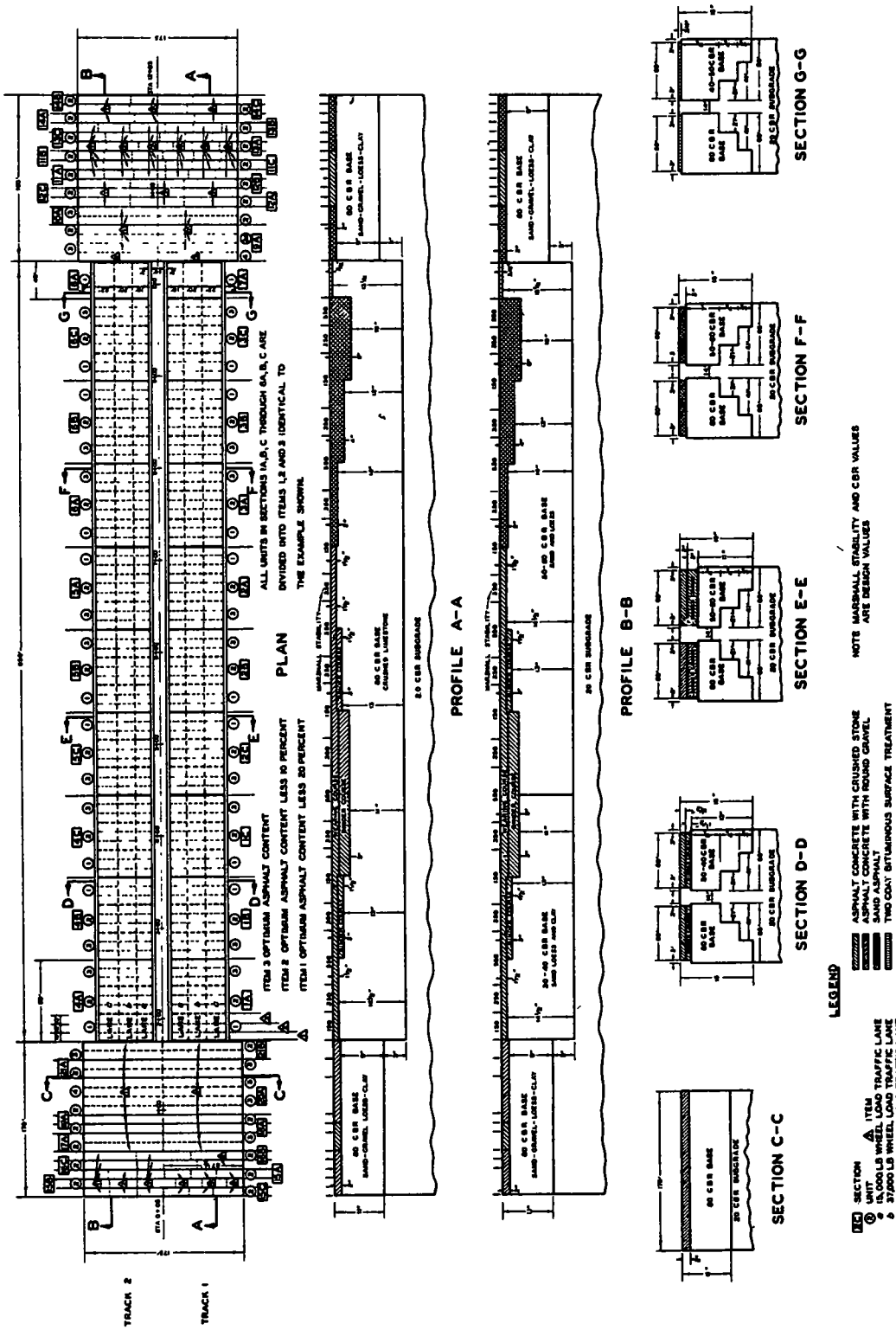


Figure 1. Plan and Profiles of Test Section

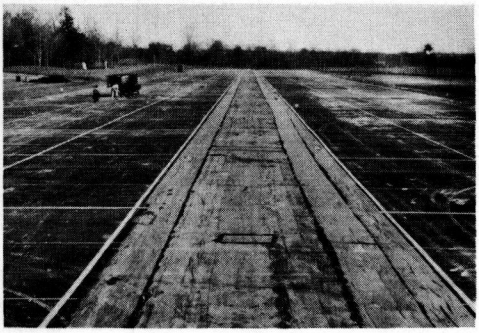


Figure 2. View of Completed Test Section

the subgrade which affected these tests. For ease of construction, the turnarounds were made 11 in. thick over the entire extent.

Base - In order to provide the necessary range, four different base materials were used. Crushed limestone was placed throughout the entire length of one test track. This base was built up in layers using procedures similar to those for waterbound macadam. No failures occurred in the crushed limestone base during traffic. Two blended base materials, designated sand-loess and sand-clay-loess, were placed in the other test track. The sand-loess consisted of 80 percent river sand and 20 percent loess. The resultant mixture had practically zero plasticity. The sand-clay-loess consisted of 60 percent washed river sand, 30 percent clayey silt, and 10 percent loess. These two base course materials were difficult to compact and it was necessary to use rubber-tired rollers with 37,000-lb. wheel loads. Failures developed in certain portions of the blended bases during traffic. The fourth base material, called turnaround gravel because it was used solely in the turnarounds, was a fairly well graded gravel with fines. It gave very good service in the traffic tests. The following table shows average liquid limit, plasticity indexes, density, and CBR value at which the base materials were evaluated upon completion of the traffic tests. (Top, Page 31.)

Variables included in test section - The test tracks were divided into eight sections in each of which was a major variation in the base or wearing course. Three base materials and four types of pavement were used. Designation of the sections and the variations in each are found on page 31, center table.

In order to establish the minimum thickness of asphalt pavements for the three wheel loads, each of the sections, except the two surfaced with double-surface treatments, was subdivided into three subsections each 90 ft. long. A different thickness of pavement was placed in each subsection. Designations of the subsections and the pavement thickness in each are given on page 31, lower table.

In this symposium the subsection designation is combined with the section number and the combined symbol hereafter designates a section of the test tracks.

To determine the stability values of asphalt pavements satisfactory for the three wheel loads, each subsection was divided into three units, each 30 ft. long. A different stability value for the pavement was used in each unit. Designations of the units and stability values used in each are as follows:

Unit	Design Stability
1	Low (Marshall value of approximately 150 lb.; no filler added)
2	Medium (Marshall value of approximately 350 lb.; some filler added)
3	High (Marshall value of approximately 550 lb.; high amount of filler added)

In order to determine the relationship between the optimum asphalt content as determined by the Marshall stability test and the optimum required by traffic compaction by the three wheel loads, each unit was divided into three items 10 ft. long. A different asphalt content with respect to optimum was used in each item. Experience gained from previous use of the test procedure (test section at Marietta, Georgia) indicated that the optimum as determined by the Marshall stability test would approach the maximum permissable for traffic use; therefore, all variations in asphalt content were made on the low

Base	Liquid Limit	Plasticity Index	% Mod. AASHO Density	CBR
Crushed limestone	NP	NP	*	80+
Turnaround gravel	26	5	96	60
Sand-clay-loess	22	7	95	50
Sand-loess	NP	NP	99	40

*Characteristic compaction curve could not be determined.

Section	Base Material	Pavement
1	Crushed limestone	Asphaltic concrete, limestone aggregate
2	Crushed limestone	Asphaltic concrete, gravel aggregate
3	Crushed limestone	Sand asphalt
7	Crushed limestone	Double surface treatment
4	Sand-clay-loess	Asphaltic concrete, limestone aggregate
5	Sand-loess	Asphaltic concrete, limestone aggregate
6	Sand-loess	Sand asphalt
8	Sand-loess	Double surface treatment

	Pavement Thickness	
	Asphaltic Concrete	Sand Asphalt
Subsection	Sections 1, 2, 4, and 5	Sections 3 and 6
A	Surface course, 1½-in.	2 in.
B	Surface course, 1½-in. Binder course, 1½-in.	4 in. (2 equal courses)
C	Surface course, 2 in. Binder course, 3 in.	6 in. (2 equal courses)

side of the Marshall optimum. Item designations and variations in asphalt content are presented below.

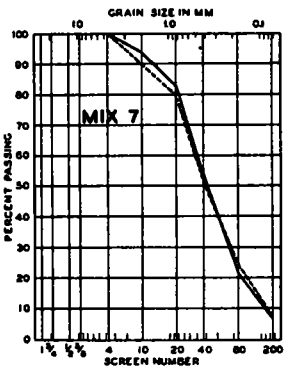
Item	Asphalt Content With Respect to Marshall Optimum*
1	20 percent below optimum
2	10 percent below optimum
3	Optimum

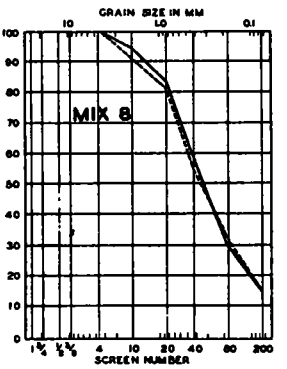
*Determined as that amount of asphalt which indicated maximum stability based on specimens compacted with 15 blows of a 10-lb. hammer 18-in. drop, one side only, plus 5,000-lb. static load held for two minutes.

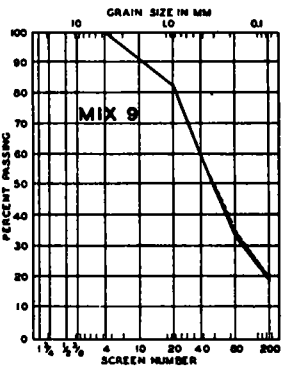
Although the turnarounds were intended primarily to provide a means of turning the heavy traffic equipment, they were also used in the general study. Only one type of base and one thickness of pavement (2 in.) were used in the turnarounds, but different mixtures with varying physical characteristics, stability, and asphalt content were used. Accordingly, the turnarounds were divided into 25 sections for the different types of pavement. Where feasible, these sections were subdivided similarly to those in the straightaways.

Asphaltic mixtures - Three basic types of paving mixtures were used in the pavements of the test section. These were asphaltic concrete in which the coarse aggregate was a 3/4-in. maximum size crushed limestone, asphaltic concrete in which the coarse aggregate was a 3/4-in. maximum size uncrushed gravel, and sand asphalt which, as is implied, contained principally fine (sand) aggregate. The fine aggregate in the majority of cases consisted of a blend of washed, siliceous sand from a sand and gravel pit blended with a fine sand from a Mississippi River sand bar. A commercial limestone dust was used as filler in all mixes. Asphalt cement, penetration grade 120 to 150, was used in all pavements.

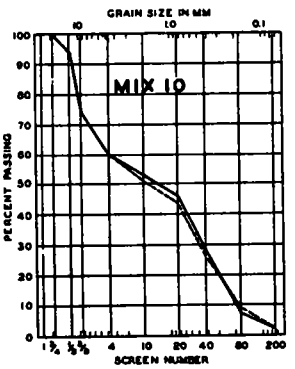
To fulfill the requirements of the test section it was necessary to use 27 differ-

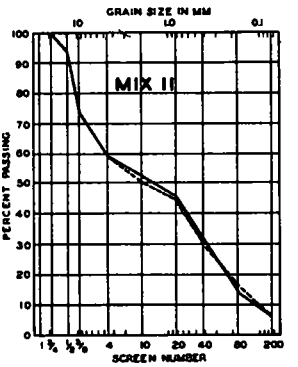


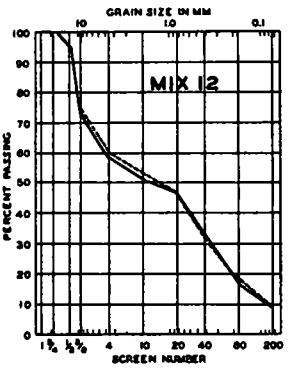




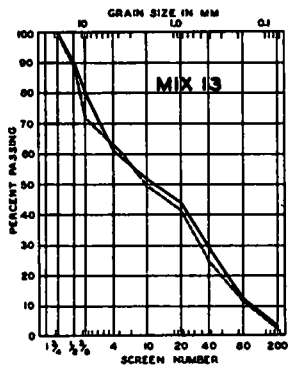
SAND ASPHALT WEARING COURSE

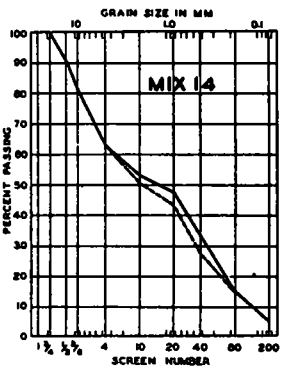


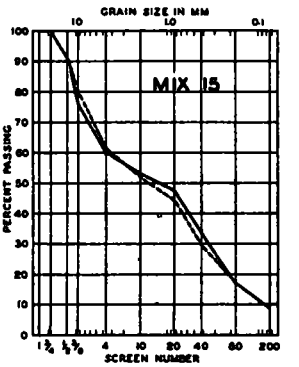




ASPHALTIC CONCRETE WEARING COURSE
CRUSHED LIMESTONE AGGREGATE







ASPHALTIC CONCRETE WEARING COURSE
UNCRUSHED LIMESTONE AGGREGATE

NOTES

GRADATION CURVES FOR LABORATORY DESIGN MIXES SHOWN BY DASH LINES.

VALUES OBTAINED IN CONSTRUCTION SHOWN BY SOLID LINES.

GRADATION CURVES
MIXES 7–15 INCLUSIVE

Figure 3

ent mixes which have been designated numerically. Mixes 1-6 were asphaltic concrete binder courses. Mixes 7, 8, and 9 were sand asphalts, and mixes 10-15 were asphaltic concretes. Mixes 7-15 comprised the wearing courses in the main test tracks and assume primary importance in the analyses in this report. Gradation curves for these mixes are shown on Figure 3, and a typical pavement core for each of the three types of aggregate is shown on Figure 4. Mixes 16-27 were placed in limited quantities in the turnarounds. Characteristics and location of each of the mixes are shown in table, page 34.

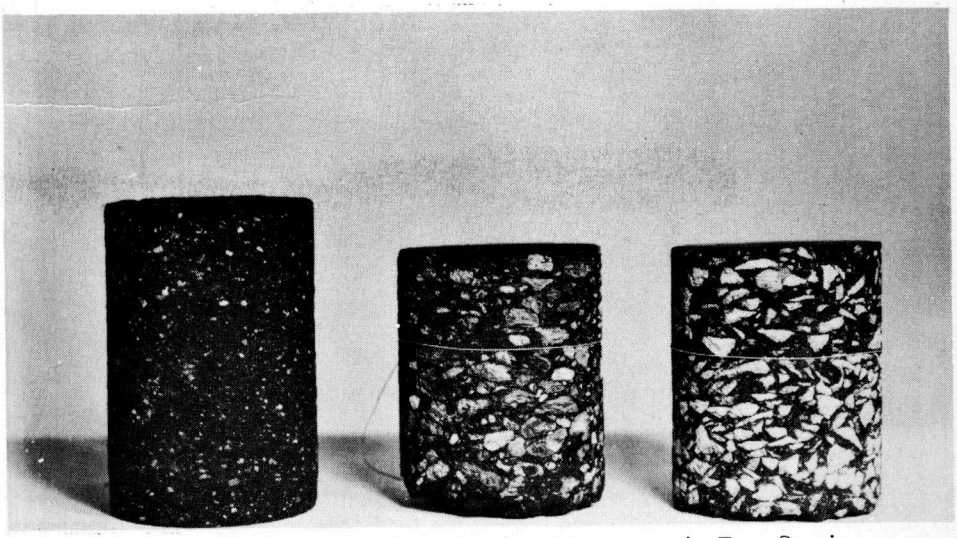


Figure 4. Typical Cores from Completed Pavement in Test Section
Note: Line shows Boundary Between Wearing and Binder Courses

The pavements listed were all obtained from the basic aggregate types described above. For asphaltic concrete, the coarse aggregate, both crushed limestone and uncrushed gravel, was about 50 percent of the total aggregate weight. Variations in stability were achieved by varying the percent of the filler. No limestone filler was added to the low stability mixes, a moderate amount (sufficient to bring the total to about 5 to 7 percent) was added for the medium stability, and sufficient to bring the total to about 10 percent was added for the high stability mixes. The same method was followed to vary the stability of the sand asphalt, except the total filler in the sand mixes was higher, about 7, 13, and 19 percent, respectively, for low, medium, and high stability mixes. In the turnarounds gap-graded asphaltic concrete mixtures were obtained by deleting the material from one plant bin. For the gap-graded sand, a poorly-graded river bar sand was used. The stone-filled sand asphalts, which were mixtures with a low percentage of coarse aggregate, were obtained by reducing the coarse aggregate to about 25 percent of the total. Also, a few mixes in the turnarounds were varied as to the amount of filler included as shown in the table.

All asphalt mixes were prepared in a standard hot-mix plant and transported to the project in trucks. The material was placed using a standard Barber-Greene spreader in 10-ft. lanes at right angles to the center line of the test section. Compaction was accomplished with a 10-ton tandem steel roller.

TESTING PROCEDURE

In these tests, traffic designated as "warm weather", was applied only when the pavement temperature was in excess of 90 F. because it was desired to apply traffic

Mix No.	Aggregate	Stability Range	Location*
	Asphaltic Concrete Binder Course		
1, 2, 3	Limestone	Low, Medium, high	1BC, 4BC, 5BC
4, 5, 6	Gravel	Low, medium, high	2BC
	Sand Asphalt		
7, 8, 9	---	Low, medium, high	3ABC, 6ABC, 9A 13ABC, 14ABC
16, 17	Gap-graded	Medium, high	10A
22, 23	25 percent and 15 percent filler	Extremely high, medium high	9A
	Asphaltic Concrete Wearing Course		
10, 11, 12	Limestone	Low, medium, high	1ABC, 4ABC, 5ABC, 17A 11ABC, 12ABC
13, 14, 15	Gravel	Low, medium, high	2ABC, 20A, 15ABC 16BC
18, 19	Gravel, coarse aggregate gap-graded	Medium, high	21A
20, 21	Gravel, fine aggregate gap-graded	Medium, high	21B
24, 25	Gravel, 12 percent and 15 percent filler	---	20A
	Stone-filled Sand Asphalt		
26	Limestone, only 25 percent coarse aggregate	Medium	18A
27	Gravel, only 25 percent coarse aggregate	Medium	19A

*Section and subsection in test project.

when conditions would be the worst from the standpoint of the pavement displacing under traffic. In some instances temperatures as high as 140 F. were recorded in the pavement during the application of traffic, and a major part of the traffic was applied when the pavement temperature was above 100F. In addition to the warm weather traffic, a limited amount of traffic was applied during cold weather to determine the effects of cold weather traffic on the pavements, especially those with the higher percentages of filler. The only effect of the cold weather traffic on the analysis was to indicate that the maximum permissable amount of filler may be slightly less on the basis of cold weather than warm weather traffic. Detailed results of the cold weather traffic are not given.

As previously stated, it was specified that traffic should be applied with 15,000 and 37,000 lb. single wheel loads, and a 60,000-lb. dual load. A Model Super C Tournapull connected to a 12-cu. yd. scraper (see Figure 5) was used for the 15,000-lb. wheel load tests. The scraper of this unit was loaded in such a manner that each of the four wheels carried 15,000 lbs. Traffic was applied to produce uniform coverage across a 12-ft. width of the lane. A total of 3,500 coverages was applied. A Model Super C Tournapull connected to a specially constructed load cart (see Figure 6) was used for the 37,000 and 60,000-lb. wheel load tests. This cart was constructed in such a manner that 56-in. diameter single or dual wheels could be mounted in the center of the cart and the weight adjusted in the

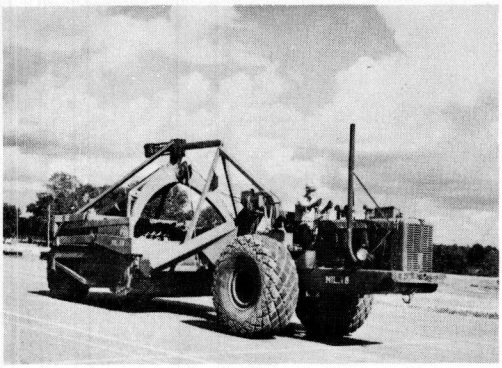


Figure 5. View of Model Super C Tournapull Connected to 12 Cu. Yd. Carryall Used for 15,000-lb. Wheel Load Traffic Tests

load boxes on each end of the cart to produce the desired wheel load. It may be seen that in addition to the main load wheel in the center of the cart there are four other wheels on the load cart and towing unit; the two wheels of the towing unit and the two wheels on the stabilizing

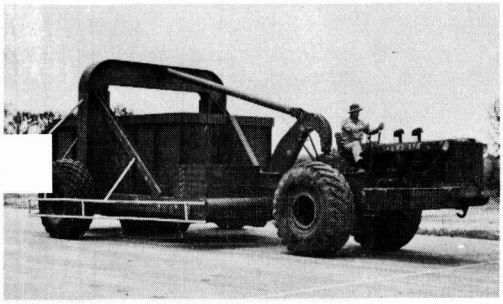


Figure 6. View of Model Super C Tournapull Connected to Specially Constructed Load Cart Used for 37,000-lb. Single and 60,000-lb. Dual Wheel Load Traffic Tests

yoke of the load cart. The Tournapull towing unit wheels each carried approximately 14,000 lbs. while the stabilizing wheels on the yoke each carried approximately 10,000 lbs. The load on these wheels was small in comparison with the test load, and their effect has been considered negligible. The 37,000-lb. wheel load traffic was applied to produce uniform coverages over a 4-ft. width and the 60,000-lb. dual wheel load over a 6-ft. width of their respective lanes. A total of 1,500 coverages was applied with each load. Figure 7 shows tire print data for the three wheel loads.

The performance of each pavement item under traffic was recorded throughout the test in the form of visual observations. The observations have been standardized into groups listed as tire printing, rutting and shoving, cracking, settlement, roughness, upheaval, and longitudinal movement (see Figures 8-13 for typical views of several of the conditions). Each of these observations was qualified as to degree, so that one of four notations was possible for each: "none," "faint," "well defined," or "pronounced." The performance of each pavement item subjected to the traffic in the test was recorded for comparison in this manner and is presented in detail in Appendix D of Waterways Experiment Station TM 3-254 *(1)*. Table 1 is a typical sheet of these observations. In addition to these observations, movements of the pavement surface were recorded by level measurements at intervals during the traffic tests. A few deflection gages were installed in order to obtain factual data relative to the amount of surface deflection that was produced by the several wheel loads. The deflection recorded by the gages was that normally expected in flexible pavements.

Pavement cores were obtained from within the traffic lanes prior to the traffic testing and at frequent intervals throughout traffic testing. Cores of 4 in. diameter and of a height equal to the pavement depth were obtained in triplicate for laboratory testing. From these cores the stability, flow, and the unit weight total mix in lb. per cu. ft. were obtained. The other test properties, unit weight aggregate only, percent voids aggregate only, percent voids total mix, and percent total voids filled with asphalt were computed. The complete record of test properties for each pavement item during the traffic tests is listed in Waterways Experiment Station TM 3-254 *(1)*. Table 2 is a typical tabulation of data.

In addition to the pavement tests, a number of inspection pits were excavated

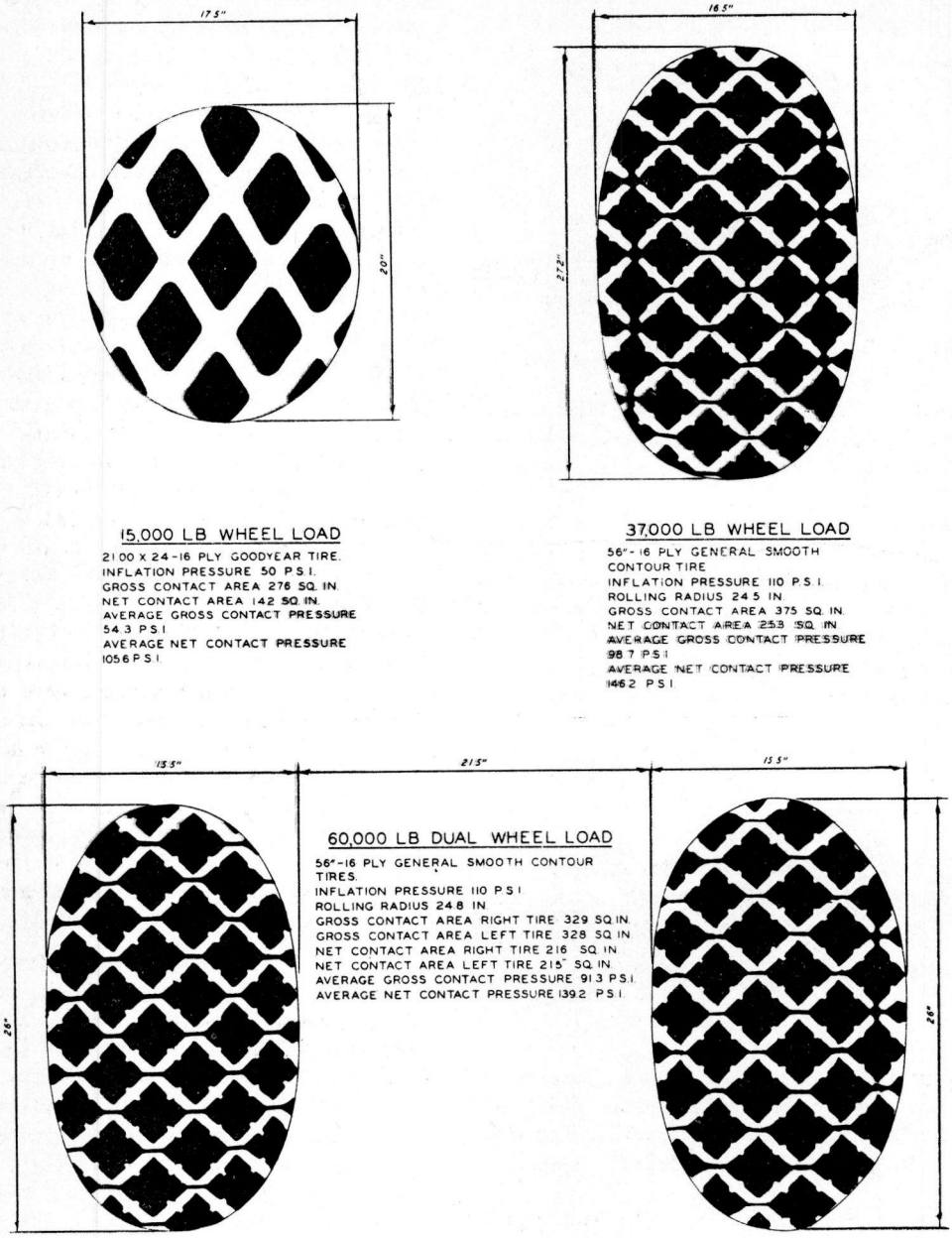


Figure 7. Tire Print Data

into the base and subgrade both during and after completion of traffic testing. Test measurements included density, moisture, and CBR tests, the results of which are not presented here.

ANALYSIS AND DISCUSSION

Evaluation of Mixes and Pavements

In the analysis of the data from the test section it was necessary to study the behavior of the various items under traffic and determine (a) which mixes were or were not satisfactory from the standpoint of resisting displacement under traffic, and (b) which pavements were or were not satisfactory from the standpoint of preventing shear deformation in the underlying base. Evaluations for these conditions were made using the observa-

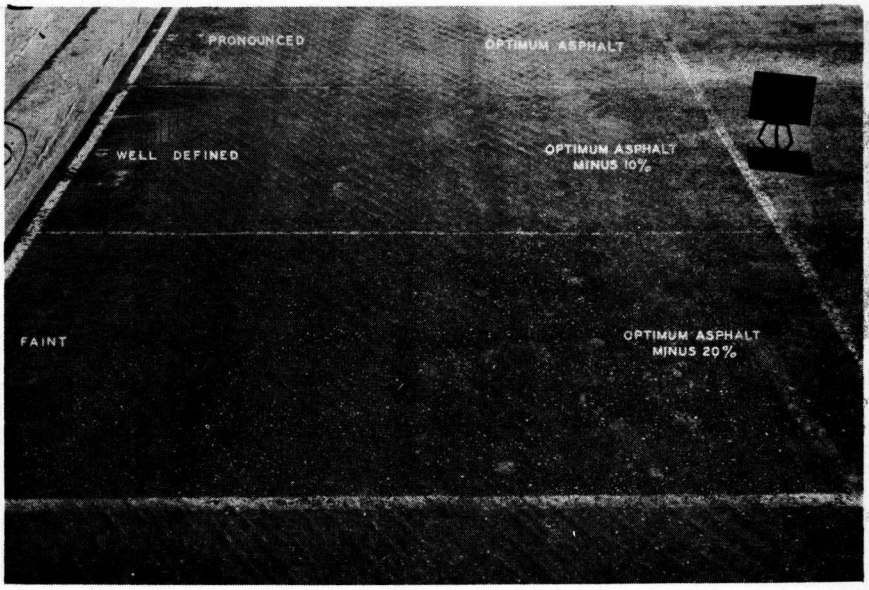


Figure 8. View Showing Typical Tire Printing, Section 2B-2, Mix 14, 2024 Coverages, 15,000-lb. Lane

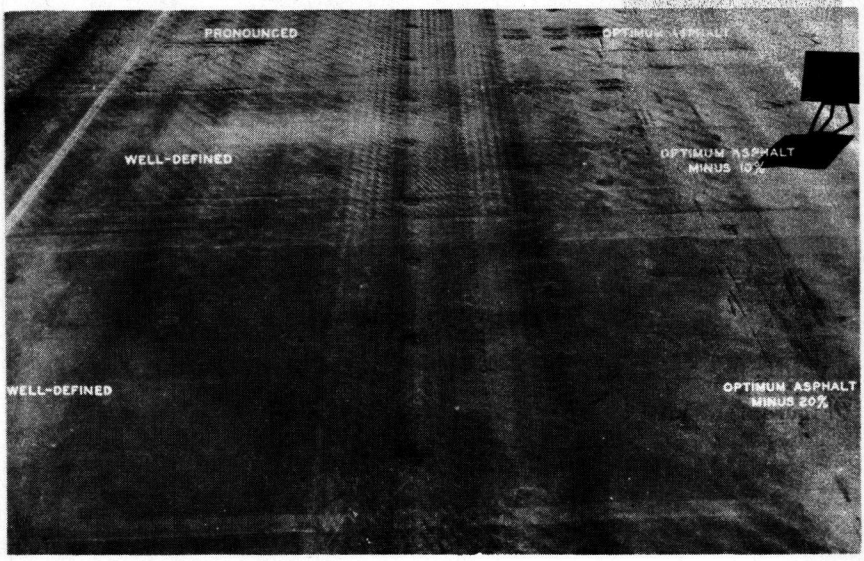


Figure 9. View Showing Typical Rutting, Section 3C-1, Mix 7, 122 Coverages, 37,000-lb. Lane

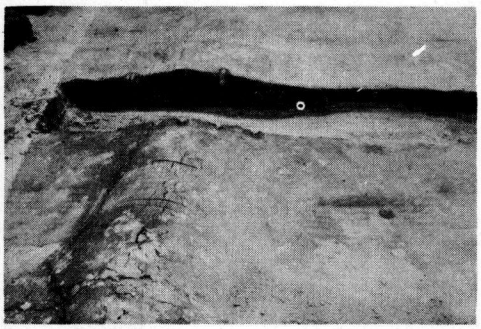


Figure 10. View Showing Pronounced Upheaval in Pavement Section 6B-1, Mix 7 Optimum Asphalt, 620 Coverages, 15,000-lb. Lane

tions and measurements during traffic as "tools."

Evaluation of Mixes - Observations of tire printing, rutting and shoving, upheavals, and surface cracking, were used in evaluating the capacity of the mixes to resist displacement under traffic. A definite significance was attached to each of the standardized adjective ratings (none, faint, well-defined, pronounced) for each of these observations. The determination of the significance attached to each of these ratings was based on field observations and was generally agreed on by all concerned, including personnel of the Office, Chief of Engineers and the consultants. Complete details and discussions of the significance attached to each rating are given in Appendix D of Waterways Experiment Station TM 3-254 *(1)* but they are too voluminous to be repeated here. For illustration, the significance attached to the

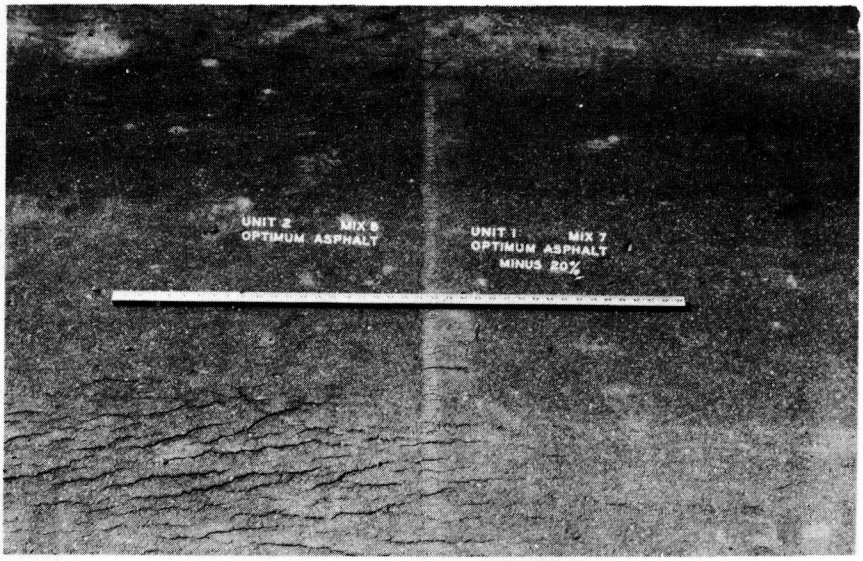


Figure 11. View Showing Surface Cracking, Section 3B, Mixes 7 and 8, 1500 Coverages, 37,000-lb. Lane

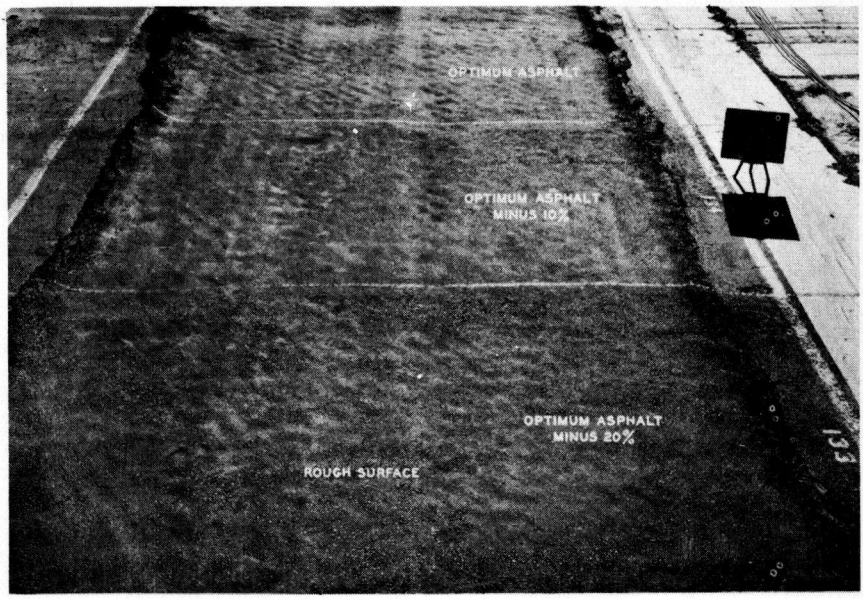


Figure 12. View Showing Pronounced Case of Condition Designated as "Rough Surface." Photo also Shows Upheaval at Sides of Traffic Lane Section 5A-1, Mix 10, 2024 Coverages, 15,000-lb. Lane

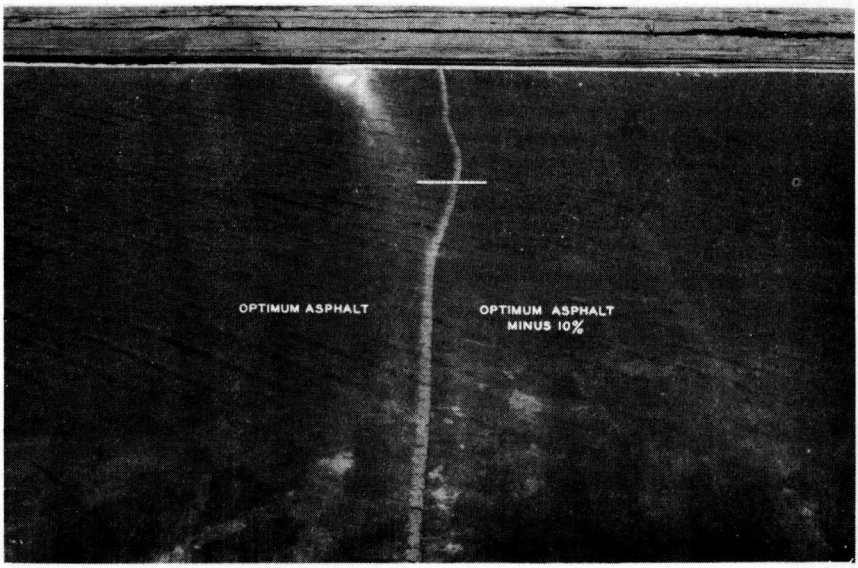


Figure 13. View Showing Pronounced Longitudinal Movement, Section 6A-1, Mix 7, 200 Coverages, 15,000-lb. Lane

TABLE 1[a]

OBSERVATIONS OF PAVEMENT BEHAVIOR
WARM WEATHER TRAFFIC TESTS

						Number of Coverages																
						Faint					Well-defined					Pronounced						
Section	Unit	Item	Asphalt Content %	Pavement Thickness in	Wheel Load lb	Tire Printing	Rutting and Shoving	Upheaval	Cracking	Settlement	Tire Printing	Rutting and Shoving	Upheaval	Cracking	Settlement	Tire Printing	Rutting and Shoving	Upheaval	Cracking	Settlement	Remarks
												Mix No 11									
1A	2	1	4 8	1 5	15,000	1300	None	None	None	1500	None	None	None	None	None	None	None	None	None	None	
					37,000	None	900	None	1300	100	None	None	None	1500	700	None	None	None	None	1300	
					60,000	100	None	None	None	400	None	None	None	None	1300	None	None	None	None	None	
		2	5 4	1 5	15,000	1300	None	None	None	1500	None	None	None	None	None	None	None	None	None	None	
					37,000	100	800	None	400	100	None	None	None	None	1000	None	None	None	None	1300	
					60,000	100	600	None	None	400	None	None	None	None	1300	None	None	None	None	None	
		3	6 0	1 5	15,000	200	None	None	3000	1500	300	None	None	None	None	None	None	None	None	None	
					37,000	1000	None	None	400	100	None	None	None	None	700	None	None	None	None	1300	
					60,000	100	600	None	None	1300	200	None	None	None	1500	None	None	None	None	None	
1B	2	1	4 8	1 5	15,000	None	None	None	None	1400	None	None	None	None	None	None	None	None	None	None	
					37,000	None	None	None	None	100	None	None	None	None	700	None	None	None	None	None	
					60,000	None	None	None	None	1500	None	None	None	None	None	None	None	None	None	None	
		2	5 4	1 5	15,000	1300	None	None	None	1400	None	None	None	None	None	None	None	None	None	None	
					37,000	None	800	None	None	100	None	None	None	None	1000	None	None	None	None	None	
					60,000	None	None	None	None	700	None	None	None	None	None	None	None	None	None	None	
		3	6 0	1 5	15,000	300	1200	None	None	1100	500	None	None	None	None	None	None	None	None	None	
					37,000	None	100	None	None	100	None	None	None	None	1000	None	None	None	None	None	
					60,000	100	600	None	None	1500	None	None	None	None	None	None	None	None	None	None	
1C	2	1	4 8	2 0	15,000	1200	3000	None	None	3500	None	None	None	None	None	None	None	None	None	None	
					37,000	100	100	None	1200	600	None	None	None	None	1000	None	None	None	None	None	
					60,000	200	1500	None	None	1500	None	None	None	None	None	None	None	None	None	None	
		2	5 4	2 0	15,000	500	1300	None	None	3500	1200	None	None	None	None	None	None	None	None	None	
					37,000	None	100	None	1200	100	None	None	None	None	1000	None	None	None	None	None	
					60,000	200	1500	None	None	1500	None	None	None	None	None	None	None	None	None	None	
		3	6 0	2 0	15,000	200	400	None	None	1700	500	1600	None	None	None	None	None	None	None	None	
					37,000	100	None	None	1200	300	None	None	None	None	700	None	None	None	None	None	
					60,000	200	500	None	1500	700	None	1500	None	None	None	None	None	None	None	None	
4A	2	1	4 8	1 5	15,000	None	None	None	None	600	None	None	None	None	None	None	None	None	None	None	
					37,000	None	300	1500	800	100	None	500	None	-	700	None	None	None	1500	1500	Faint roughness and longitudinal movement Incipient failure at 1500 coverages
					60,000	None	700	1500	None	600	None	None	None	None	1300	None	None	None	None	1500	Faint roughness and longitudinal movement
		2	5 4	1 5	15,000	200	None	None	None	1500	None	None	None	None	None	None	None	None	None	None	
					37,000	200	100	1500	1500	100	None	None	None	None	700	None	None	None	None	1100	Surface of pavement uneven Faint roughness and longitudinal movement
					60,000	200	600	1500	None	600	None	None	None	None	1500	None	None	None	None	None	Surface of pavement uneven Faint roughness and longitudinal movement
		3	6 0	1 5	15 000	200	400	3500	None	600	None	1000	None	None	None	None	None	None	None	None	
					37,000	300	100	-	500	100	1500	None	1500	None	700	None	None	None	None	1000	Surface of pavement uneven Well-defined roughness and faint longitudinal movement
					60,000	200	600	-	1400	600	None	1500	1500	None	1500	None	None	None	None	None	Surface of pavement uneven Well-defined roughness and faint longitudinal movement

[a]This is a sample page from Table 1 for illustrative purposes

TABLE 2[a]

RESULTS OF LABORATORY TEST ON FIELD CORES OBTAINED DURING TRAFFIC TESTS

15,000-Lb Lane

Section	Unit	Item	Asphalt Cement %	Wearing Course Thickness ins	Coverage	Marshall Stability Lbs.	Flow Value Units of 1/100 Inch	Unit Weight Lbs Per Cu Ft Total Mix	Unit Weight Lbs Per Cu Ft Aggregate Only	Voids Percent Total Mix	Voids Percent Aggregate Only	Total Voids Filled With Asphalt %
						Mix No. 10 (Continued)						
5B	1	1	5.6	1.5	0	172	15	141 1	133 2	9 3	21.7	57.2
					312	291	13	143 2	135.2	7 9	20 5	61.5
					1452	502	11	146.5	138 3	5.8	18 7	69.0
					2100	943	11	147.9	139 6	4.9	17.9	72.7
					3500	810	14	145 7	137 5	6 3	19 1	67.1
		2	6.3	1.5	0	229	13	143.1	134.1	7 1	21.3	66.6
					312	470	15	145.8	136.6	5 3	19.7	73.1
					1452	759	15	147.7	138.4	4.1	18.7	78.1
					2100	950	14	147.8	138 5	4.0	18.6	75 5
					3500	1002	14	148.2	138 9	3 7	18.4	79 9
		3	7 0	1.5	0	367	14	144 6	134 5	5.1	21 0	75.7
					312	556	16	144 9	134 8	4 8	20.7	76.9
					1452	713	20	147.4	137.1	3.2	19 0	83.1
					2100	1152	21	148 6	138 2	2.4	18.7	87.2
					3500	781	18	148 3	137.9	2 6	18.9	80.3
5C	1	1	5 6	2	0	80	17	140.5	132.6	9.6	22 0	56.3
					312	247	14	144.6	136.5	7.1	19.8	64.2
					1540	331	10	145.7	137.5	6.3	19 0	66.8
					2100	596	11	147.2	139.0	5.4	18 4	70.6
					3500	628	15	146 8	138.6	5.6	18.5	69.8
		2	6.3	2	0	170	17	144.5	135 4	6.2	20.5	69.7
					312	175	13	147.2	137.9	4.4	19 0	76 8
					1540	533	13	147 1	137.8	4.4	19.0	76.8
					2100	847	13	148.5	139.1	3.6	18.3	80.3
					3500	967	14	149.3	139.9	3.1	17.9	82.7
		3	7 0	2	0	245	16	144.7	134 6	5.0	20.9	76.1
					312	415	15	146.2	136.0	4 0	20.1	80 1
					1540	654	18	146 2	136.0	4.0	20 1	80.1
					2100	698	17	147.4	137.8	3 2	19.4	83 5
					3500	755	17	148 3	137.9	2.7	19.0	85.8
						Mix No. 11						
1A	2	1	4.8	1.5	0	349	15	144 6	137 7	8.3	19.2	56.8
					136	666	17	148.1	141 0	5 8	17.0	65 8
					520	809	13	149.4	142 2	5.2	16.5	68.4
					800	837	14	149 1	141 9	5.5	16.8	67.2
					1652	997	12	150.4	143 2	4.6	15.9	71.1
					2100	1381	12	150 7	143 5	4.5	15.9	71.6
					3500	1440	13	150.1	142.9	4.9	16.2	69.8
		2	5.4	1.5	0	375	15	145 1	137.3	7 2	19 5	63.1
					136	568	16	147.8	139 8	5 4	17.9	69.9
					520	846	14	149.7	141 6	4.2	16.9	75.1
					800	972	13	149 7	141.6	4.3	17 0	74.7
					1652	1287	13	150 3	142.2	3.8	16.6	77.0
					2100	1347	12	150 2	142.1	3.9	16 6	76.6
					3500	1253	16	150.7	142 6	3.6	16.4	78 0
		3	6.0	1 5	0	482	17	146.5	137 7	5 4	19.2	71.9
					136	611	18	148.1	139.2	4 3	18.3	76.5
					520	767	17	150.0	141 0	3.1	17.2	82.0
					800	826	15	149 4	140.4	3 5	17.6	80.1
					1652	1091	17	150.5	141 5	2.8	17 0	83.5
					2100	1303	16	149.7	140 7	3.3	17.4	81 0
					3500	1032	18	150.3	141 3	2.9	17 1	83.0

[a]This is a sample page from Table 2 for illustrative purposes

four degrees of tire printing is given.

a. *None*. Mix may be approaching an unsatisfactory condition from the standpoint of brittleness.

b. *Faint*. Not considered detrimental, in fact faint tire printing may be beneficial.

c. *Well-defined*. Not considered detrimental.

d. *Pronounced*. Considered an indication that the mix is approaching an unsatisfactory condition from the standpoint of displacing under traffic (plastic) but is not considered sufficiently detrimental to warrant eliminating an otherwise satisfactory mix.

tic," and "plastic." Figures 14 and 15 show typical evaluations, and table 3 lists the evaluations. In this table mixes with the same aggregates are grouped together. Separate evaluations are made for items in the main tracks and the turnarounds. The behavior of any given asphalt content for a mix is indicated by its place under the column headings. For example Mix 11, the ninth entry in the table, was placed at asphalt contents ranging from 3.0 to 8.0 percent although in the main track the range was only from 4.8 percent to 6.0 percent. In the main track, Mix 11 was evaluated as "approaching brittle" when the asphalt content was

Figure 14. View Showing Typical Mix Evaluation, Section 2A-3, Mix 15, 2024 Coverages, 15,000-lb. Lane

On the basis of the significance attached to the observations, each mix at each asphalt content was evaluated. Individual items were evaluated initially, but it was determined that thickness of pavement, wheel load, and type of underlying base were not factors: therefore, all items of the same mix number and asphalt content were evaluated as a group. Four evaluations were used: "approaching brittle," "satisfactory," "border plas-

4.8 percent, as "satisfactory" when the asphalt content was 5.4 percent, and "border plastic" at 6.0 percent asphalt content. The evaluations in the turnarounds are indicated in a similar manner. The last column in table 3 shows the range of acceptable asphalt contents. This range is based only on the items in the main track. In the study of the behavior, there was relatively little information on brittleness; hence, it was

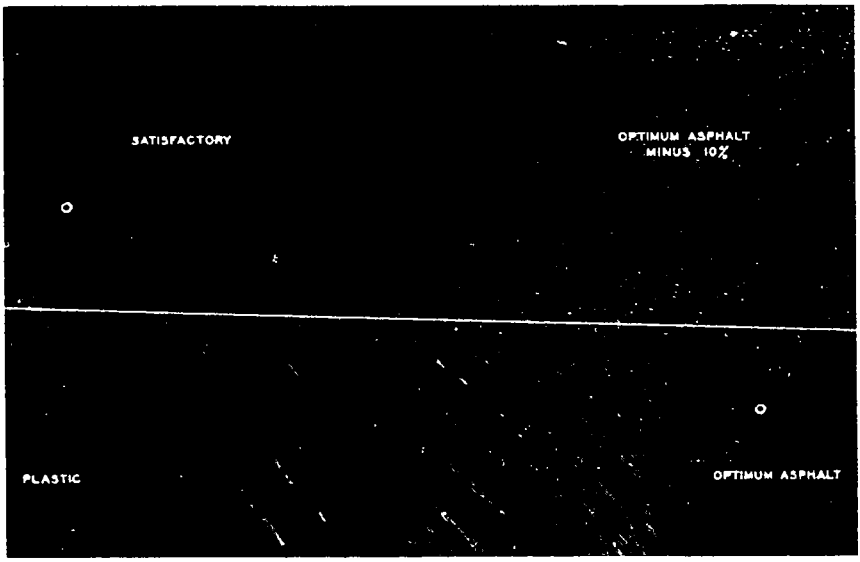


Figure 15. View Showing Typical Satisfactory and Plastic Mixes
Section 2A-1, Mix 13, 1100 Coverages, 37,000-lb. Lane

TABLE 3

EVALUATION OF MIXES

Material	Mix Number	Optimum Asphalt Content %	Section	Asphalt Content at Indicated Behavior: Approaching Brittle, Main Tracks	Approaching Brittle, Test Tracks	Approaching Brittle, Turnarounds	Satisfactory, Main Tracks	Satisfactory, Test Tracks	Satisfactory, Turnarounds	Border Plastic, Main Tracks	Border Plastic, Test Tracks	Border Plastic, Turnarounds	Plastic, Main Tracks	Plastic, Test Tracks	Plastic, Turnarounds	Acceptable Range of Asphalt Contents
Sand Asphalt	7	10 6	3, 6	-		-	-		-	8 5		-	9 5-10 6		-	7 6-8 5
	8	8 8	3, 6, 9, 13, 14	-		5 0	-		6 0	7 0		7 0	7 9-8 8		7 9-10 0	6 3-7 0
	9	8 0	3, 6, 9	-		-	6 4		6 4	7 2		7 2	8 0		-	6 4-7 2
	23	6 8	9	-		-	-		6 0-6 8	-		-	-		-	-
	22	8 3	9	-		-	-		-	-		-	-		7 5-8 3	-
Sand Asphalt (Gap-graded)	16	9 0	10	-		-	-		8 0	-		-	-		9 0	-
	17	7 5	10	-		-	-		6 7	-		7 5	-		-	-
Asphaltic Concrete (Crushed Limestone)	10	7 0	1, 4, 5	5 6		-	6 3		-	7 0		-	-		-	6 3-7 0
	11	6 0	1, 4, 5, 11, 12, 17	4 8		3 0 -4 0	5 4		4 8-5 8	6 0		6 0-7 0	-		8 0	5 4-6 0
	12	6 0	1, 4, 5, 17	-		-	4 8-5 4		4 8	-		5 4	6 0		-	4 8-5 4
Asphaltic Concrete (Uncrushed Gravel)	13	7 5	2	-		-	6 0-6 8		-	-		-	7 5		-	6 0-6 8
	14	7 1	2, 15, 16, 20	-		-	5 7-6 4		5 0-6 4	-		6 5-6 6	7 1		7 0-8 0	5 7-6 4
	15	5 5	2, 20	4 5		-	5 0-5 5		4 5-5 0	-		-	-		-	5 0-5 5
	24	4 5	20	-		-	-		4 0-4 5	-		-	-		-	-
	25	5 1	20	-		-	-		4 4-5 1	-		-	-		-	-
Asphaltic Concrete (Uncrushed Gravel Gap-graded)	18	5 7	21	-		-	-		5 1-5 7	-		-	-		-	-
	19	5 4	21	-		-	-		4 8-5 4	-		-	-		-	-
	20	6 0	21	-		-	-		5 4-6 0	-		-	-		-	-
	21	5 0	21	-		-	-		4 5-5 0	-		-	-		-	-
Asphaltic Concrete (25% Coarse Aggregate)	26	7 7	18	-		-	-		-	-		6 8	-		7 7	-
	27	7 7	19	-		-	-		-	-		6 8	-		7 7	-
Binder Courses	1, 2, 3, 4, 5, 6	-	-	-		-	All		-	-		-	-		-	-

not possible to pick out definitely brittle mixes except in the turnarounds. There were certain indications (primarily lack of tire printing) of mixes having brittle characteristics; therefore, mixes having these characteristics are classified as "approaching brittle." The classification "satisfactory" includes those which did not have detrimental behavior. It should be realized that this includes mixes with a range of asphalt contents and from the standpoint of durability the ones with the higher asphalt contents may be more satisfactory. The classification "border plastic" includes mixes which are approaching a rich mix. They represent the maximum asphalt contents that could be allowed in the test section without having a detrimental displacement. The classification "plastic" includes mixtures which are definitely rich.

Evaluation of Pavements - Observations of rutting and shoving, upheaval, cracking (other than surface cracking), rough surface, and longitudinal movement were used in evaluating the capacity of the pavement to prevent shear deformation in the underlying base. In addition, profiles of trenches cut across the traffic lanes were used in identifying when shear deformation had occurred. In general the same procedures were used for evaluating the pavements as for evaluating the mixes using a definite significance for each of the adjective ratings. The significance attached to the adjective ratings of the varying degrees of "rough surface" is given below as an example.

a. *None*. A good indication that the pavement is adequate to prevent shear deformation in the underlying base.

b. *Faint*. Not considered sufficiently detrimental to warrant labeling an otherwise satisfactory pavement as unsatisfactory on the basis of this observation alone, but it does indicate that the pavement is approaching the condition of being inadequate to prevent shear deformation in the underlying base material.

c. *Well-defined*. Pavement is not quite adequate to prevent shear deformation in the underlying base material.

d. *Pronounced*. Pavement is definitely inadequate to prevent shear deformation in the underlying base material.

By analyzing the observations as a whole it was possible to evaluate the different pavements on the basis of their ability to prevent shear deformation in the underlying base. Clear-cut evaluations of "satisfactory" or "unsatisfactory" could be made in many cases; in others it was necessary to evaluate the pavement as "border-line." The wheel load, thickness, and type of underlying base were definite factors in this analysis and individual evaluations for specific items of pavement were necessary in many instances. Table 4 shows the evaluations. No evaluations are presented for pavements on the crushed limestone or the turnaround gravel as no shear deformation occurred in these bases. On table 4, the asphalt contents are given in percent of Marshall optimum (at peak of stability curve). The actual asphalt contents at this optimum are listed in the third column of table 3. Asphalt contents at 20 percent and 10 percent below optimum can be computed readily (for Mix 11, optimum was 6.0 - 20 percent below was 4.8 percent and 10 percent below was 5.4 percent).

Effect of Traffic on Test Properties

In the following paragraphs the changes that took place in the test properties during traffic are studied and the data obtained on the mixtures are analyzed to determine pertinent test property values for use in selecting limiting criteria and for other analyses.

At the time the test section was constructed and at intervals throughout the traffic testing, cored samples were removed from each item in the section for laboratory tests. These tests consisted of the measurement of the stability, flow, and density of each specimen. The magnitude of this phase of the work can be best appreciated by a review of the work involved. There were 486 items in the two main tracks. Each item was cored from four to seven times, and three specimens were removed at each interval. Assuming

TABLE 4[a]

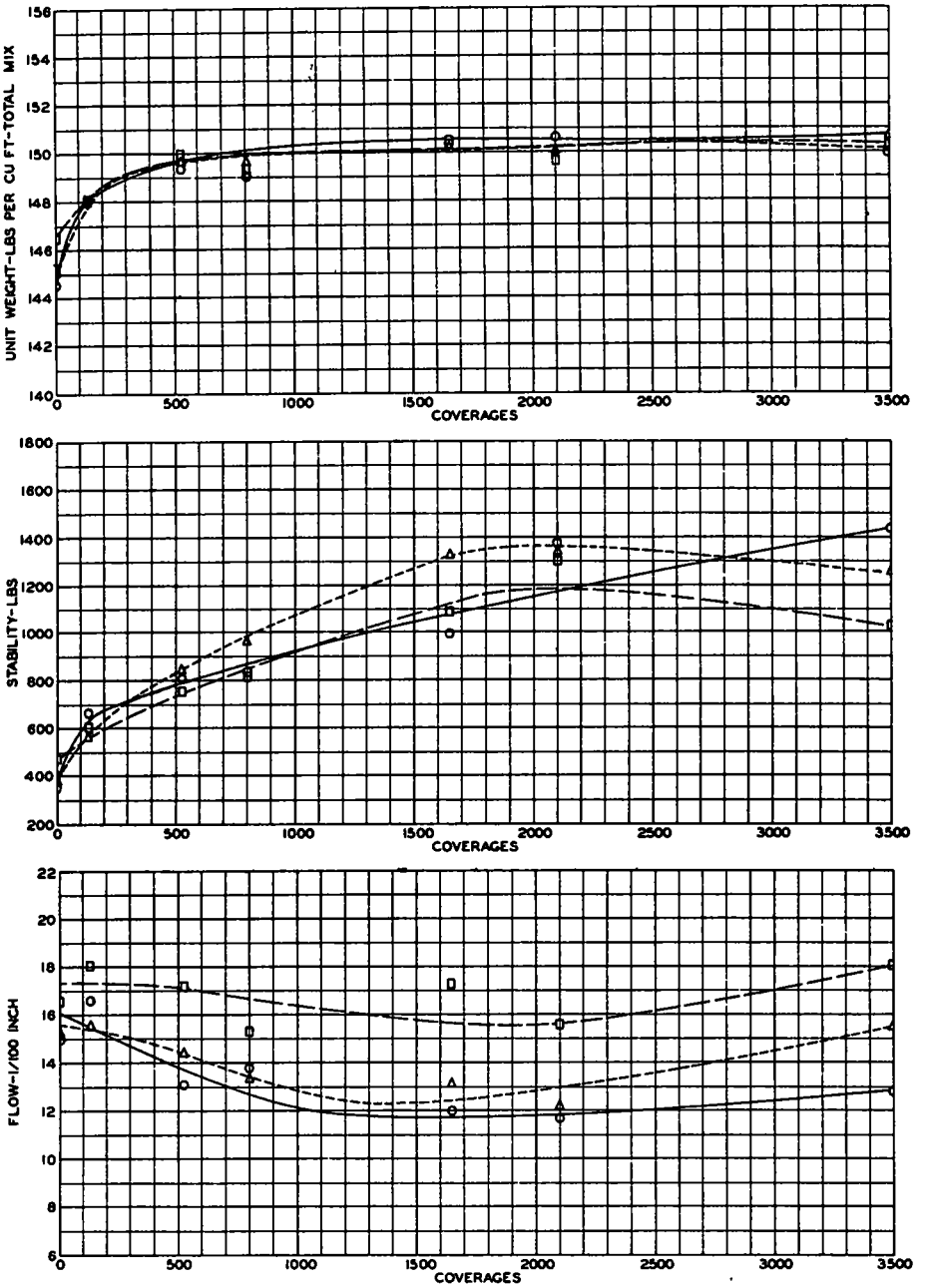
PAVEMENT EVALUATION - INFERIOR BASES

Section	Unit	Type Base	Pavement Thickness Inches	Wheel Load Kips	Mix Number	Evaluation at Indicated Asphalt Content[b] Marshall Optimum-20%	Marshall Optimum-10%	Marshall Optimum
4A	1	Sand-clay-loess	1 5	15	10	S	S	S
	1			37	10	B	B	U
	1			60	10	B	B	U
4A	2	Sand-clay-loess	1 5	15	11	S	S	S
	2			37	11	B	B	U
	2			60	11	B	B	U
4A	3	Sand-clay-loess	1 5	15	12	S	S	B
	3			37	12	B	B	U
	3			60	12	B	U	U
4B	1	Sand-clay-loess	3 0	15	10	S	S	S
	1			37	10	S	S	B
	1			60	10	S	S	B
4B	2	Sand-clay-loess	3 0	15	11	S	S	S
	2			37	11	S	B	U
	2			60	11	S	B	U
4B	3	Sand-clay-loess	3 0	15	12	S	S	S
	3			37	12	B	U	U
	3			60	12	B	U	U
4C	All	Sand-clay-loess	5 0	15, 37, 60	10, 11, 12	S	S	S
5A	All	Sand-loess	1 5	15, 37, 60	10, 11, 12	U	U	U
5B	1	Sand-loess	3 0	15	10	U	U	U
	1			37	10	U	U	U
	1			60	10	U	U	U
5B	2	Sand-loess	3.0	15	11	S	B	U
	2			37	11	S	U	U
	2			60	11	S	U	U
5B	3	Sand-loess	3 0	15	12	S	S	B
	3			37	12	S	S	U
	3			60	12	S	S	U
5C	1	Sand-loess	5 0	15	10	S	B	U
	1			37	10	S	B	U
	1			60	10	S	S	S
5C	2	Sand-loess	5 0	15	11	S	S	S
	2			37	11	S	S	S
	2			60	11	S	S	S
5C	3	Sand-loess	5 0	15	12	S	S	S
	3			37	12	S	S	S
	3			60	12	S	S	S
6A	All	Sand-loess	2 0	15, 37, 60	7, 8, 9	U	U	U
6B	All	Sand-loess	4 0	15, 37, 60	7, 8, 9	U	U	U
6C	1, 2	Sand-loess	6 0	15, 37, 60	7, 8	U	U	U
6C	3	Sand-loess	6 0	15	9	B	U	U
				37	9	B	U	U
				60	9	B	U	U
8A	1	Sand-loess	0 75	15, 37, 60	Surface treated	U	U	U

[a]This is a sample page from Table 4 for illustrative purposes
[b]Abbreviations S - Satisfactory, B - Borderline, U - Unsatisfactory

15 cores were taken from each item, more than 7,000 were subjected to laboratory test. If the turnaround items are included, the total exceeds 10,000 specimens. In order to reduce this volume of statistical data to reasonable proportions for analytical purposes, it was necessary that the data be summarized. The steps taken to condense the data for one mix are presented in the following paragraphs.

The basic data showing the results of the laboratory tests on the pavement cores for each test item are presented in tables of which table 2 is a sample sheet (out of 66). These results are the average of at least three test specimens, representing the first step in summarizing the data. Mix 11, which is an asphaltic concrete containing about 50 percent crushed limestone as the coarse aggregate fraction, is used to illustrate the method whereby the data were further summarized. On



LEGEND

O———O ITEM 1-OPTIMUM ASPHALT CONTENT LESS 20%*4.8% ASPHALT CEMENT

Δ- - - - - - Δ ITEM 2-OPTIMUM ASPHALT CONTENT LESS 10%*5.4% ASPHALT CEMENT

□— — — □ ITEM 3-OPTIMUM ASPHALT CONTENT*, 6.0% ASPHALT CEMENT

NOTE. *OPTIMUM ASPHALT CONTENT DETERMINED BY ORIGINAL MARSHALL LABORATORY PROCEDURE

TRAFFIC COMPACTION DATA

MIX NO 11 - 15,000 - LB LANE
SEC 1A -UNIT 2 -ITEMS 1,2 AND 3

Figure 16

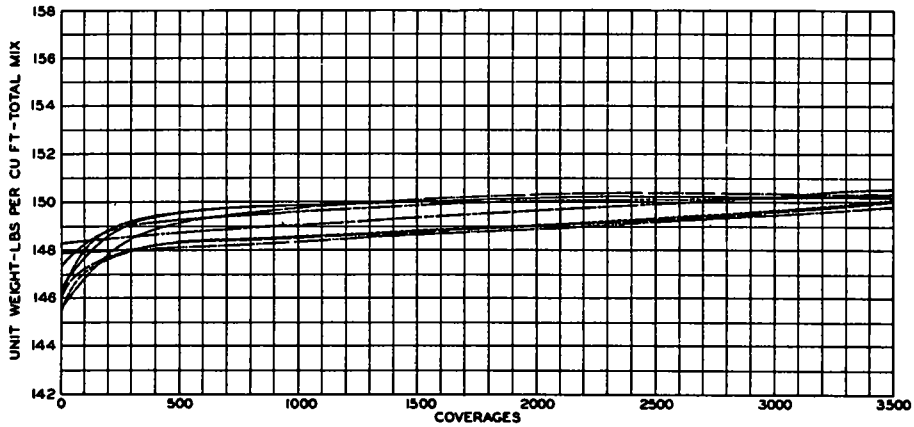
Figure 16 the data summarized in table 2 representing the results for Mix 11, optimum asphalt, 1½-in. pavement, are plotted, and curves are drawn approximately through the plotted points. (Flow values are plotted to nearest 10th, values in table have been rounded off.) Also included on this figure are curves obtained in the same manner for those items of Mix 11, 1½-in. pavement that contained minus 10 and minus 20 percent asphalt. The variation in three test properties - stability, flow, and unit weight total mix - is shown for the various pavement mixtures with the application of coverages of a 15,000 lb. wheel load. Since the other density and void properties are computed from the unit weight total mix, they would reflect the same changes and need not be included at this time. Similar plots were made for Mix 11 from other locations in the 15,000-lb. wheel load lane. It should be remembered that Mix 11 was placed in the 1½-, 3-, and 5-in. sections on the crushed rock base, the sand-clay base, and the sand-clay-loess base so that in all, Mix 11 appears in nine locations in the 15,000-lb. wheel load lane.

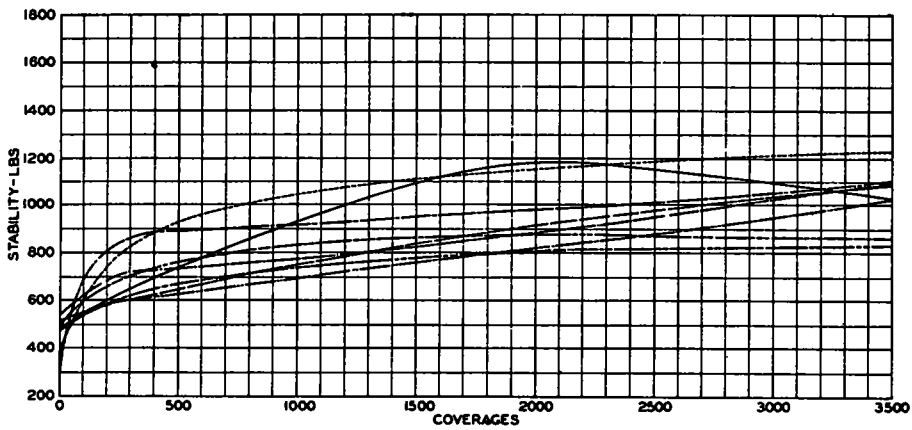
Figure 17 shows, for all but one location, changes in stability, flow, and unit weight total mix, respectively, with coverages for Mix 11 at 6.0 percent asphalt. On Figure 17 it can be seen that the unit weight total mix varied between 146 and 148 lbs. at the time of construction for the eight items shown. The unit weight total mix of all items increased with coverages and at the end of 3500 coverages was 150 lbs. with a tolerance of one-half lb. plus or minus. A similar comparison of unit weight total mix is also possible for Mix 11 at other asphalt contents. At all three asphalt contents an average curve would include with one or two exceptions all curves if a tolerance of plus or minus one lb. is permitted. The same comparisons are made for the curves for stability and flow. In the case of stability, similar trends are indicated for all curves at a given asphalt content. However, there appears to be a wider scattering of the data. Since a test of this type will inevitably show variation, it is considered that the curves are in reasonable agreement and an average curve would in general be satisfactory. Some of the flow curves on Figure 17 show somewhat different trends from the others; however, the flow curves for the other asphalt contents for Mix 11 are more consistent. For this reason, it is believed that for the purpose of developing a general case, an average flow value is satisfactory.

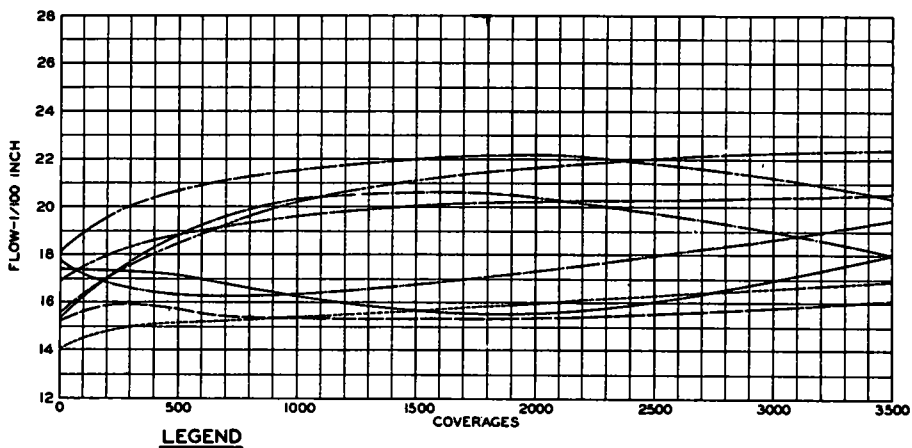
The summation of all the data for Mix 11 in the 15,000-lb. lane is presented on Figure 18. On this figure the difference produced by a change in asphalt content is demonstrated. Each of the curves represents the weighted average of eight items (one of the nine original items was deleted because base failure occurred). Each item was cored about six times with three specimens being secured each time. This represents a total of approximately 150 test specimens; therefore, the average curves developed in this manner are considered reliable. This procedure was followed to establish the average curves for Mixes 7 through 15 and in each of the three wheel load lanes.

From the summaries of the data as plotted on Figure 18, it is possible to secure the points required to develop the several test property curves in which asphalt content is plotted as one of the coordinates. Curves of the various test properties versus asphalt content may be drawn for any given number of coverages during traffic testing. Figure 19 shows curves for a range of coverages for Mix 11 in the 15,000-lb. lane. Similar curves were plotted for the other mixes. With the data summarized as shown on Figures 18 and 19 it is possible to study the changes that occurred in the test properties during traffic. These changes are discussed in the following paragraphs.

Stability - On Figure 18 the variation in stability with coverages is shown for three asphalt contents. As constructed, the pavement items placed at optimum asphalt (determined in the preliminary design to be 6 percent) and the items placed at 10 percent less than optimum asphalt (5.4 percent) both had an average stability







LEGEND

———	SECTION 1-A	—··—	SECTION 4-B
- - - - -	SECTION 1-B	—···—	SECTION 4-C
— — —	SECTION 1-C	— — —	SECTION 5-B
—·—	SECTION 4-A	— — —	SECTION 5-C

NOTE SECTION 5A OMITTED BECAUSE ALL ITEMS IN THIS SECTION FAILED PRIOR TO THE END OF TRACKING

*OPTIMUM ASPHALT CONTENT DETERMINED BY ORIGINAL MARSHALL LABORATORY PROCEDURE

TRAFFIC COMPACTION DATA

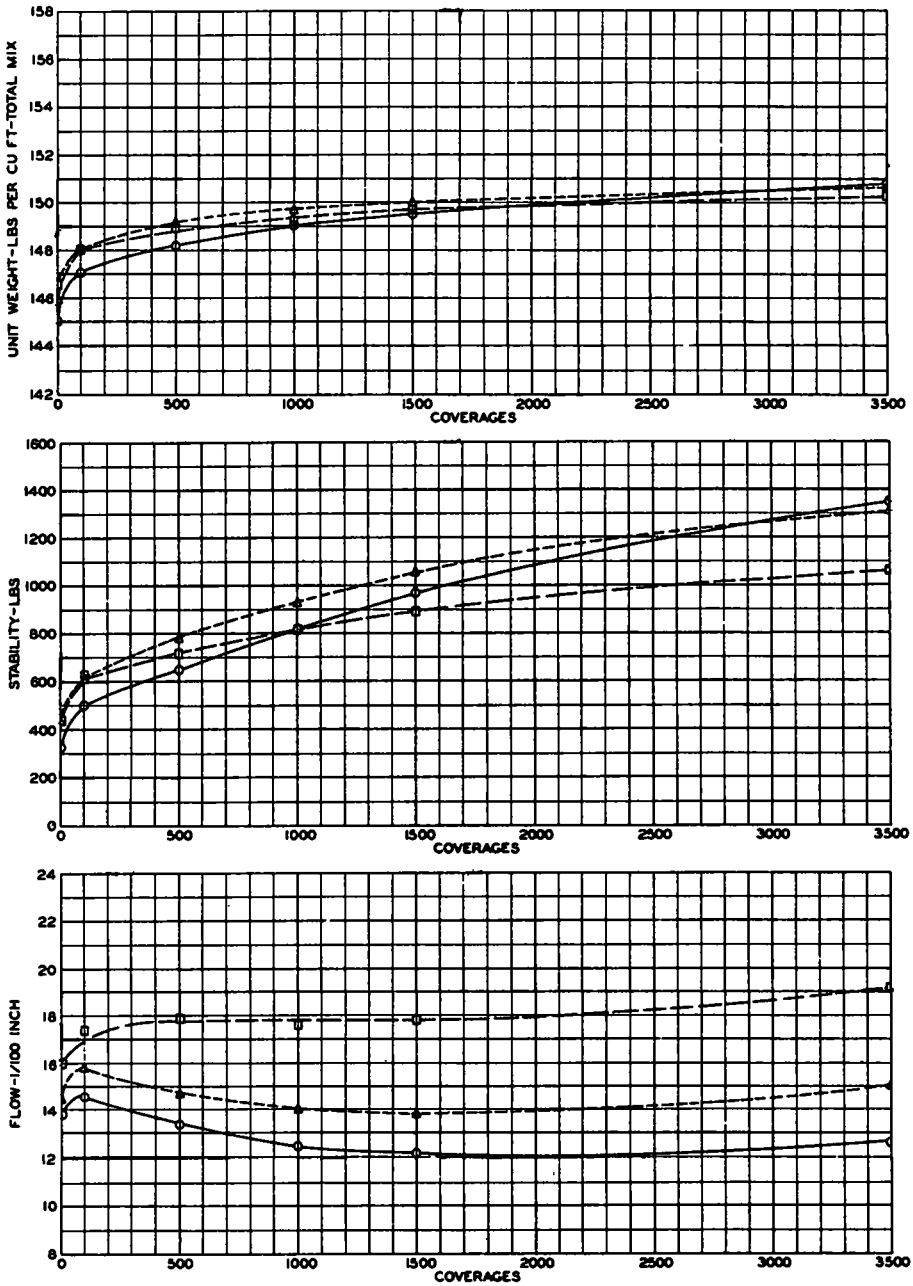
MIX NO 11 - 15,000 -LB LANE

SUMMARY CURVES

OPTIMUM ASPHALT CONTENT

*6.0 % ASPHALT CEMENT

Figure 17



LEGEND

O———O ITEM 1-OPTIMUM ASPHALT CONTENT LESS 20% 4.8% ASPHALT CEMENT

Δ-------Δ ITEM 2-OPTIMUM ASPHALT CONTENT LESS 10% 5.4% ASPHALT CEMENT

□— — —□ ITEM 3-OPTIMUM ASPHALT CONTENT*, 6.0% ASPHALT CEMENT

NOTE *OPTIMUM ASPHALT CONTENT DETERMINED BY ORIGINAL MARSHALL LABORATORY PROCEDURE

TRAFFIC COMPACTION DATA

MIX NO 11 - 15,000 -LB LANE
AVERAGE CURVES

Figure 18

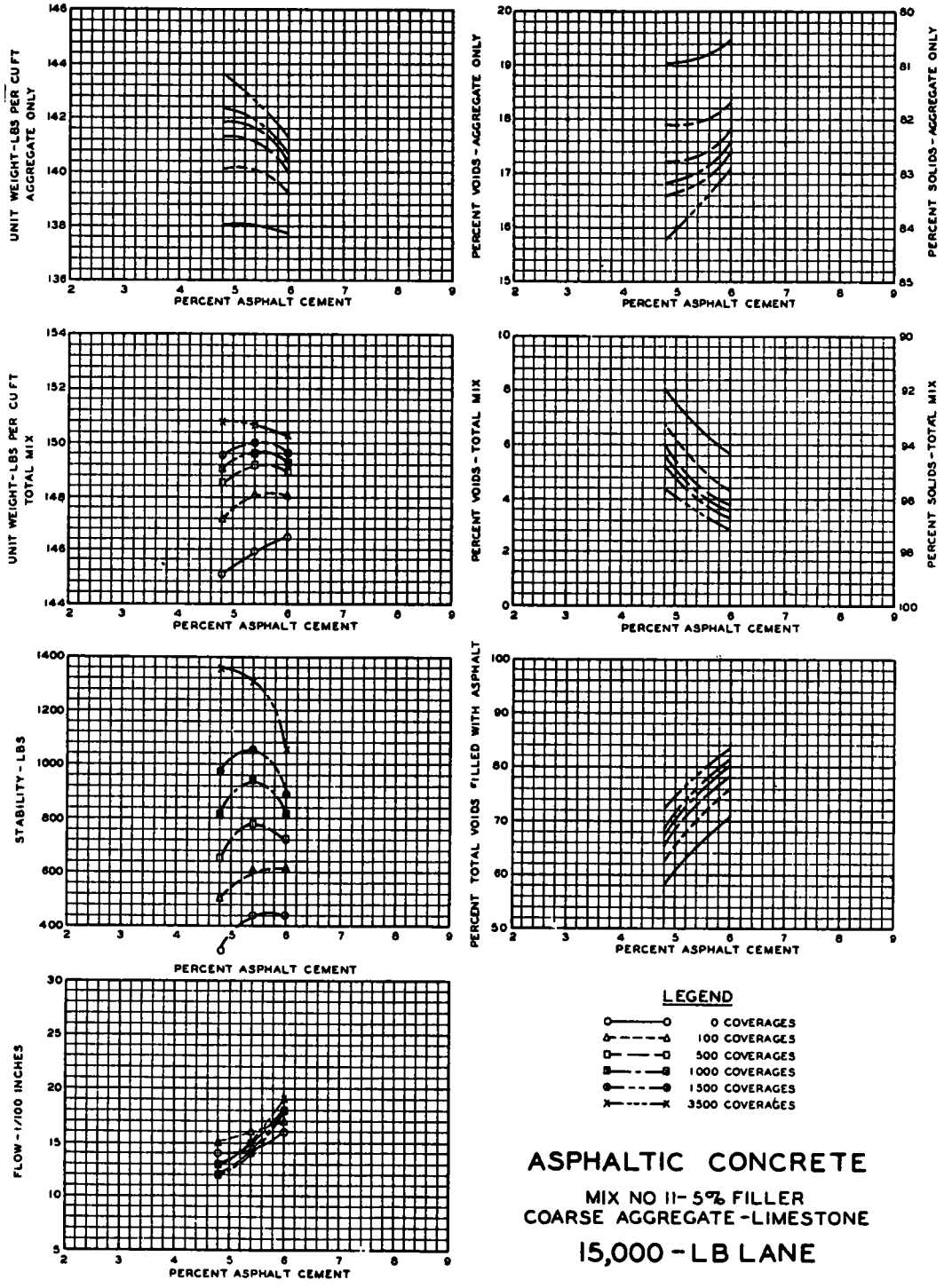

UNIT WEIGHT-LBS PER CU FT AGGREGATE ONLY
PERCENT ASPHALT CEMENT
PERCENT VOIDS - AGGREGATE ONLY
PERCENT SOLIDS - AGGREGATE ONLY
UNIT WEIGHT-LBS PER CU FT TOTAL MIX
PERCENT VOIDS - TOTAL MIX
PERCENT SOLIDS - TOTAL MIX
STABILITY - LBS
PERCENT TOTAL VOIDS FILLED WITH ASPHALT
FLOW - 1/100 INCHES
LEGEND
0 COVERAGES
100 COVERAGES
500 COVERAGES
1000 COVERAGES
1500 COVERAGES
3500 COVERAGES
ASPHALTIC CONCRETE
MIX NO 11- 5% FILLER
COARSE AGGREGATE - LIMESTONE
15,000 - LB LANE

Figure 19

of 440 lbs. Items placed at 20 percent less than optimum asphalt had the lowest stability, averaging about 330 lbs. as constructed. As the pavement items were subjected to coverages of a 15,000-lb. wheel load, the stability of all items at the three asphalt contents increased constantly to 3,500 coverages. The items that contained minus 20 percent asphalt increased in stability more rapidly than the others so that they not only equalled but exceeded items with optimum and minus 10 percent asphalt. The items containing optimum asphalt increased the least, so that at the end of 3,500 coverages the order of the items from lowest to highest was completely reversed from their order at the start of the tests.

Flow - The flow values of the three asphalt contents for Mix 11 as constructed were 14, 15, and 16 for minus 20, minus 10, and optimum asphalt content, respectively. These variations are considered negligible. As can be seen on Figure 18, the pavement items with optimum asphalt increased slightly in flow and at the end of 3,500 coverages amounted to 19, or a gain of three points. Items placed at minus 10 percent asphalt changed scarcely at all during the entire test and were 15 upon completion of tracking. The minus 20 percent asphalt items actually decreased somewhat and had a flow of 13 at the end of test. The observations made of their performance under the action of the 15,000-lb. wheel load show that the items containing optimum asphalt had well-defined or pronounced tire prints, showed indications of very slight rutting and shoving, and otherwise gave indications that the asphalt content was slightly high. The fact that the flow increased with traffic is believed to be significant. The items containing minus 10 percent asphalt performed satisfactorily under traffic without appreciable movement or marking. It is pointed out that the flow in this case neither increased nor decreased with traffic coverages. The items that contained minus 20 percent asphalt also were entirely satisfactory and did not move under traffic. Since the flow decreased slightly, it is suggested that the amount of asphalt was a trifle deficient and the pavement was becoming more brittle with coverages.

Unit weight-total mix - As shown on Figure 18, at the time of construction the items placed at optimum asphalt had the greatest density, 146.5 lbs; those placed at minus 10 percent were next at 145.9 lbs; and items placed at minus 20 percent had the least density, 145.0 lbs. As was the case with stability, the total weight for all items increased with the application of traffic coverages but at different rates for the three asphalt contents. Upon completion of 3,500 coverages, the density of the items at the two lower asphalt contents was nearly identical at 150.7 and 150.8 lbs. per cu. ft., while optimum asphalt items had increased to only 150.3 lbs. In round numbers the gain in density amounted to 6, 5, and 4 lbs. per cu. ft., respectively, for the minus 20, minus 10, and optimum items. The curves were still showing a slight increase at the completion of the test, but it is believed that the increase in density with additional coverages would have been negligible.

Test properties versus asphalt content - On Figure 19 the changes that take place with coverages of a 15,000-lb. wheel load on pavements containing Mix 11 placed at three asphalt contents are shown for seven test properties. In this figure the data are shown as typical test property curves versus asphalt content at 1, 100, 500, 1000, 1500, and 3500 coverages. It is clearly shown by these curves that if optimum asphalt were to be based on the peaks of the stability or unit weight curves, the amount indicated would be reduced as the coverages increased.

Summary - Both the density and the stability of all items increased with traffic. The increase in density must result in a tighter packing of the aggregate particles and a reduction in both the aggregate voids and the air voids. This increase in the density of the aggregate resulted in greater interlock of the particles which produced higher stability at all asphalt contents. Since the voids are less at higher density, the asphalt-void ratio must also be higher in all cases.

TABLE 5*

AVERAGE TEST PROPERTY VALUES
ASPHALTIC CONCRETE MIXES - MAIN TEST TRACKS

Mix Number	Asphalt Content %	Coverages Number	Number of Items	Marshall Stability Lbs 15,000 Lb Lane	37,000 Lb Lane	60,000 Lb Lane	Flow Value Units of 1/100 Inch 15,000 Lb Lane	37,000 Lb Lane	60,000 Lb Lane	Unit Weight - Lbs Per Cu Ft Total Mix 15,000 Lb Lane	37000 Lb Lane	60,000 Lb Lane	Unit Weight Aggregate Only 15,000 Lb Lane	37,000 Lb Lane	60,000 Lb Lane	Voids - Percent Total Mix 15,000 Lb Lane	37,000 Lb Lane	60,000 Lb Lane	Voids Aggregate Only 15,000 Lb Lane	37,000 Lb Lane	60,000 Lb Lane	Voids Filled with Asphalt 15,000 Lb Lane	37,000 Lb Lane	60,000 Lb Lane	Evaluation
10	5 6	0	7	180	200	230	15	14	13	142 5	143 2	142 9	134 5	135 2	134 9	8 4	7 9	8 1	20 9	20 5	20 7	60	62	61	Approaching brittle
	(-20)	500		370	450	480	13	14	13	145 8	145 5	145 3	137 6	137 4	137 2	6 3	6 5	6 6	19 1	19 3	19 4	67	66	66	
		1,500		540	590	430	13	17	16	146 9	146 5	146 0	138 7	138 3	137 8	5 6	5 8	6 1	18 5	18 7	18 9	70	69	68	
		3,500		830			12			147 9			139 6			4 9			17 9			73			
	6 3	0	7	200	270	270	15	14	13	144 1	144 4	143 9	135 0	135 3	134 8	6 4	6 2	6 5	20 7	20 5	20 8	69	70	69	Satisfactory
	(-10)	500		480	620	640	13	14	14	147 4	146 9	147 5	138 1	137 6	138 2	4 3	4 6	4 2	18 9	19 1	18 8	77	76	78	
		1,500		720	720	550	13	18	16	148 3	148 4	148 0	139 0	139 1	138 7	3 7	3 6	3 9	18 3	18 3	18 5	80	80	79	
		3,500		1110			14			149 2			139 8			3 1			17 9			83			
	7 0	0	7	270	270	300	16	14	16	144 3	144 3	144 6	134 2	134 2	134 5	5 3	5 3	5 1	21 2	21 2	21 0	75	75	76	Border Plastic
	(Opt)	500		520	640	650	17	16	17	146 3	147 4	146 8	136 1	137 1	136 5	4 0	3 4	3 6	20 1	19 6	19 8	80	83	82	
		1,500		670	710	620	18	22	17	147 3	148 3	147 4	137 0	137 9	137 1	3 3	2 6	3 2	19 5	18 9	19 4	83	86	84	
		3,500		860			18			148 2			137 8			2 7			19 0			86			
11	4 8	0	8	330	400	350	14	14	14	145 0	145 5	144 4	138 0	138 5	137 5	8 1	7 8	8 5	19 0	18 8	19 4	58	58	56	Approaching brittle
	(-20)	500		650	810	830	13	14	14	148 3	147 8	148 2	141 2	140 7	141 1	6 0	6 3	6 1	17 2	17 5	17 3	65	64	65	
		1,500		970	870	840	12	16	19	149 5	149 5	148 6	142 3	142 3	141 5	5 3	5 3	5 9	16 6	16 6	17 1	68	68	66	
		3,500		1350			13			150 8			143 6			4 4			15 8			72			
	5 4	0	8	440	510	410	14	14	14	145 9	146 2	144 9	138 0	138 3	137 1	6 7	6 5	7 3	19 1	18 9	19 6	65	66	63	Satisfactory
	(-10)	500		780	970	960	15	15	14	149 2	148 9	149 5	141 1	140 9	141 4	4 6	4 8	4 4	17 3	17 4	17 1	73	73	74	
		1,500		1050	1010	960	14	18	15	150 0	150 5	149 9	141 9	142 4	141 8	4 1	3 8	4 2	16 8	16 6	16 9	76	77	75	
		3,500		1310			15			150 7			142 6			3 6			16 4			76			
	6 0	0	8	440	500	520	16	16	17	146 5	146 6	146 7	137 7	137 9	137 9	5 7	5 3	5 3	19 5	19 1	19 1	71	72	72	Border plastic
	(Opt)	500		720	920	880	18	17	18	148 9	149 6	149 7	140 0	140 6	140 7	3 9	3 4	3 3	17 9	17 5	17 4	78	81	81	
		1,500		890	970	900	18	21	19	149 6	150 8	150 4	140 6	141 8	141 4	3 4	2 6	2 9	17 5	16 8	17 1	81	85	83	
		3,500		1060			19			150 3			141 3			2 9			17 1			83			

*This is a sample page from Table 5 for illustrative purposes

The increase in flow is believed to indicate that the voids are becoming overfilled and is reflected by a more plastic condition of the pavement.

With the possible exception of flow, the numerical values for all test properties are variables which are dependent entirely upon the amount of compaction that has been applied. Still referring to Mix 11, the density of the items with minus 20 percent asphalt varied between 145.0 and 150.8 lbs. per cu. ft. This change can also be expressed as an increase in the percent of maximum theoretical density (percent solids) from 91.9 to 95.6 percent. To say that this particular mix was designed to have, say, 95 percent density is true only for one particular condition or time in the life of the pavement. At all other times, it is either more or less than this value. Therefore, it is apparent that if values are to be established for the test properties, they must be made with respect to a very definite compactive effort which must be specified and used to qualify such values.

Limiting Criteria

Curves similar to Figures 18 and 19 were developed for the mixes in each of the three traffic lanes. From these curves, values of the test properties were determined at intervals of traffic and were tabulated. Table 5 is a sample sheet; others were prepared for each mix. It can be noted on Figure 18 or in Table 5 that the test properties changed considerably between 0 and 500 coverages, but they do not change as rapidly between 500 and 1500 coverages. The rate of change between 1500 and 3500 coverages is even less. It was desired to select criteria which would be representative of a pavement that had been subjected to a reasonable amount of traffic. Since the changes were not great between 500 coverages and 1500 coverages and because data for 3500 coverages were not available for the 37,000-lb. single wheel and 60,000-lb. dual wheel, the test property values determined for all mixes were selected for study at 500 and 1500 coverages. It also can be seen in Table 5 that the unit weight showed very little variation in the three traffic lanes after 500 to 1500 coverages. At 1500 coverages the maximum variation in the unit weight of any mix (only two are shown on the sample sheet) under the three wheel loads was one lb. per cu. ft., while the average variation was about .5 lb. per cu. ft. As shown on Figure 7, the net contact pressures for the 15,000-, 37,000-, and 60,000-lb. wheel loads were 106, 146, and 139 psi., respectively. Within the limits of the data presented, it is shown that the total load on the tire was not a factor in producing a differential in unit weight of the pavement. It is considered that the respective test properties for the three wheel load lanes could be averaged.

Tables 6 and 7 show the summarized test properties for the various mixes. In this summary values are shown only for 500 and 1500 coverages. Also, the values for all three wheel loads have been averaged and maximum and minimum values shown. Values for unit weight have been deleted, since it was known from previous laboratory tests that they could not be used as limiting criteria. In these summaries, the data are grouped in accordance with their performance evaluation; that is, all mixes with asphalt contents which were judged to be "approaching brittle" are grouped, as are those that are "satisfactory," "border plastic," and "plastic." The "border plastic" mixes represent the extreme upper limit for satisfactory mixes. In the following paragraphs the data are analyzed to determine limiting values for flow, percent voids total mix, and percent voids filled with asphalt. Because traffic was spread over a wider area in the turnarounds than in the straight tracks, the number of coverages applied in the turnarounds was less than in the main tracks and the exact number could not be determined. For this reason it is considered that the turnaround data are less reliable than the data for the straight tracks. The range of data on asphaltic concrete from the straight tracks was adequate and the turnaround data have not been used, although they show similar

TABLE 6[a]

AVERAGE TEST PROPERTY VALUES
ASPHALTIC CONCRETE MIXES - MAIN TEST TRACKS
Summary

Mix Number	Asphalt Content %	Coarse Aggregate Type[b]	Design Stability lbs	Filler Content %	Marshall Stability Lb 0 Coverages Min	Max	500 Coverages Min	Max	1500 Coverages Min	Max	Flow Value Units of 1/100 Inch 0 Coverages Min	Max	500 Coverages Min	Max	1500 Coverages Min	Max	Voids-Percent Total Mix 0 Coverages Min	Max	500 Coverages Min	Max	1500 Coverages Min	Max	Voids-Percent Aggregate Only 0 Coverages Min	Max	500 Coverages Min	Max	1500 Coverages Min	Max	Voids-Percent Filled with Asphalt 0 Coverages Min	Max	500 Coverages Min	Max	1500 Coverages Min	Max	Evaluation
10	5.6	CL	150	2	180	230	370	480	430	590	13	15	13	14	13	17	7.9	8.4	6.3	6.6	5.6	6.1	20.5	20.9	19.1	19.4	18.5	18.9	60	62	66	67	68	70	Approaching brittle
11	4.8	CL	350	5	330	400	650	830	840	970	14	14	13	14	12	19	7.8	8.5	6.0	6.3	5.3	5.9	18.8	19.4	17.2	17.5	16.6	17.1	56	58	64	65	66	68	Approaching brittle
15	4.5	UG	550	9	310	440	800	960	840	1110	14	16	15	16	15	18	7.3	8.2	5.6	5.9	4.6	5.4	17.4	18.2	15.9	16.2	15.0	15.7	55	58	64	65	66	69	Approaching brittle
10	63	CL	150	2	200	270	480	640	550	720	13	15	13	14	13	18	6.2	6.5	4.2	4.6	3.6	3.9	20.5	20.8	18.8	19.1	18.3	18.5	69	70	76	78	79	80	Satisfactory
11	5.4	CL	350	5	410	510	780	970	960	1050	14	14	14	15	14	18	6.5	7.3	4.4	4.8	3.8	4.2	18.9	19.6	17.1	17.4	16.6	16.9	63	66	73	74	75	77	Satisfactory
12	4.8-5.4	CL	550	8	500	650	920	1150	1030	1210	15	17	15	17	15	21	5.2	7.4	3.7	5.3	3.1	4.4	17.8	18.0	16.4	16.8	15.6	16.3	60	71	68	78	71	81	Satisfactory
13	6.0-6.8	UG	150	4	240	370	550	790	480	790	13	16	15	18	15	21	4.2	6.4	2.8	4.2	2.1	3.9	19.4	19.6	17.7	18.3	16.9	18.0	67	78	76	85	78	88	Satisfactory
14	5.7-6.4	UG	350	5	290	440	660	820	650	840	14	17	15	19	15	22	4.6	6.9	2.7	4.0	2.3	3.5	18.3	19.5	16.8	17.7	16.3	17.4	65	76	77	84	79	86	Satisfactory
15	5.0-5.5	UG	550	9	530	630	840	1020	900	1220	14	17	16	19	16	22	5.1	6.0	3.3	4.0	2.8	3.6	17.2	18.3	15.4	16.2	15.1	16.0	65	71	74	79	76	82	Satisfactory
10	7.0	CL	150	2	270	300	520	650	620	710	14	16	16	17	17	22	5.1	5.3	3.4	4.0	2.6	3.3	21.0	21.2	19.6	20.1	18.9	19.5	75	76	80	83	83	86	Border plastic
11	6.0	CL	350	5	440	520	720	920	890	970	16	17	17	18	18	21	5.3	5.7	3.3	3.9	2.6	3.4	19.1	19.5	17.4	17.9	16.8	17.5	71	72	78	81	81	85	Border plastic
12	6.0	CL	550	8	600	720	770	1010	860	950	17	18	20	21	24	26	4.5	4.7	3.0	3.7	2.7	3.3	18.5	18.6	17.2	17.8	16.9	17.4	75	76	79	83	81	84	Plastic
13	7.5	UG	150	4	360	400	500	520	460	560	18	19	21	27	23	31	3.3	3.7	2.3	2.7	2.3	2.5	20.0	20.4	19.2	19.5	19.2	19.4	82	84	86	88	87	88	Plastic
14	7.1	UG	350	5	370	420	520	570	540	620	19	20	23	29	25	31	3.6	4.4	2.7	3.1	2.2	2.7	19.5	20.1	18.8	19.1	18.3	18.7	78	82	84	86	86	88	Plastic
Comparison by Type of Coarse Aggregate																																			
10-11	4.8-5.6	CL	-	-	180	400	370	830	430	970	13	15	13	14	12	19	7.8	8.5	6.0	6.6	5.3	6.1	18.8	20.9	17.2	19.4	16.6	18.9	56	62	64	67	66	70	Approaching brittle
15	4.5	UG	-	-	310	440	800	960	840	1110	14	16	15	16	15	18	7.3	8.2	5.6	5.9	4.6	5.4	17.4	18.2	15.9	16.2	15.0	15.7	55	58	64	65	66	69	Approaching brittle
10-11-12	4.8-6.3	CL	-	-	200	650	480	1150	550	1210	13	17	13	17	13	21	5.2	7.4	3.7	5.3	3.1	4.4	17.8	20.8	16.4	19.1	15.6	18.5	60	71	68	78	71	81	Satisfactory
13-14-15	5.0-6.8	UG	-	-	240	630	550	1020	480	1220	13	17	15	19	15	22	4.2	6.9	2.7	4.2	2.1	3.9	17.2	19.6	15.4	18.3	15.1	18.0	65	78	74	85	76	88	Satisfactory
10-11-12	6.0-7.0	CL	-	-	270	720	520	1010	620	970	14	18	16	21	17	26	4.5	5.7	3.0	4.0	2.6	3.4	18.5	21.2	17.2	20.1	16.8	19.5	71	76	78	83	81	86	Plastic[c]
13-14	7.1-7.5	UG	-	-	360	420	500	570	460	620	18	20	21	29	23	31	3.3	4.4	2.3	3.1	2.2	2.7	19.5	20.4	18.8	19.5	18.3	19.4	78	84	84	88	86	88	Plastic[c]
Comparison by Filler Content																																			
10	5.6	-	150	Low-2	180	230	370	480	430	590	13	15	13	14	13	17	7.9	8.4	6.3	6.6	5.6	6.1	20.5	20.9	19.1	19.4	18.5	18.9	60	62	66	67	68	70	Approaching brittle
11	4.8	-	350	Med-5	330	400	650	830	840	970	14	14	13	14	12	19	7.8	8.5	6.0	6.3	5.3	5.9	18.8	19.4	17.2	17.5	16.6	17.1	56	58	64	65	66	68	Approaching brittle
15	4.5	-	550	High-9	310	440	800	960	840	1110	14	16	15	16	15	18	7.3	8.2	5.6	5.9	4.6	5.4	17.4	18.2	15.9	16.2	15.0	15.7	55	58	64	65	66	69	Approaching brittle
10-13	6.0-6.8	-	150	Low-2-4	200	370	480	790	480	790	13	16	13	18	13	21	4.2	6.5	2.8	4.6	2.1	3.9	19.4	20.8	17.7	19.1	16.9	18.5	67	78	76	85	78	88	Satisfactory
11-14	5.4-6.4	-	350	Med-5	290	510	660	970	650	1050	14	17	14	19	14	22	4.6	7.3	2.7	4.8	2.3	4.2	18.3	19.6	16.8	17.7	16.3	17.4	63	76	73	84	75	86	Satisfactory
12-15	4.8-5.5	-	550	High-8-9	500	650	840	1150	900	1220	14	17	15	19	15	22	5.1	7.4	3.3	5.3	2.8	4.4	17.2	18.8	15.4	16.8	15.1	16.3	60	71	68	79	71	82	Satisfactory
10-13	7.0-7.5	-	150	Low-2-4	270	400	500	650	460	710	14	19	16	27	17	31	3.3	5.3	2.3	4.0	2.3	3.3	20.0	21.2	19.2	20.1	18.9	19.5	75	84	80	88	83	88	Plastic[c]
11-14	6.0-7.1	-	350	Med-5	370	520	520	920	540	970	16	20	17	29	18	31	3.6	5.7	2.7	3.9	2.2	3.4	19.1	20.1	17.4	19.1	16.8	18.7	71	82	78	86	81	88	Plastic[c]
12	6.0	-	550	High-8	600	720	770	1010	880	950	17	18	20	21	24	26	4.5	4.7	3.0	3.7	2.7	3.3	18.5	18.6	17.2	17.8	16.9	17.4	75	76	79	83	81	84	Plastic[c]

[a]This is a sample page from Table 6 for illustrative purposes

[b]Abbreviations CL - Crushed Limestone UG - Uncrushed Gravel

[c]Items classified as "Border Plastic" included with those classified as "Plastic"

TABLE 7[a]

AVERAGE TEST PROPERTY VALUES
SAND ASPHALT MIXES

Summary

Mix Number	Asphalt Content %	Design Stability lbs	Filler Content %	Marshall Stability Lb 0 Coverages Min	Max	500 Coverages Min	Max	1500 Coverages Min	Max	Flow Value Units of 1/100-Inch 0 Coverages Min	Max	500 Coverages Min	Max	1500 Coverages Min	Max	Voids-Percent Total Mix 0 Coverages Min	Max	500 Coverages Min	Max	1500 Coverages Min	Max	Voids-Percent Aggregate Only 0 Coverages Min	Max	500 Coverages Min	Max	1500 Coverages Min	Max	Voids-Percent Filled with Asphalt 0 Coverages Min	Max	500 Coverages Min	Max	1500 Coverages Min	Max	Evaluation
8	5 0	350	13	130[b]		490[b]		1140[b]		21[b]		18[b]		18[b]		16 4[b]		12 4[b]		11 5[b]		26 5[b]		23 0[b]		22 2[b]		38[b]		46[b]		48[b]		Approaching brittle
8	6 0	350	13	150	150	440	560	710	1200	18	20	18	20	17	20	13 5	13 8	9 7	10 7	8.6	9 4	25 8	26 1	22 6	23 4	21 6	22 3	47	48	54	57	58	60	Satisfactory
9	6 4	550	19	350	650	670	1040	920	1120	18	29	16	21	15	23	7 0	11.8	5 1	8 3	4 0	6 0	21 1	25 2	19 5	22 2	18 6	20 3	53	67	63	74	70	78	Satisfactory
16	8 0	350	14	210	330	400	520	450	950	20	24	23	25	25	27	7 1	8 0	4 4	5 1	3 5	4 7	24 4	25 1	22 2	22 7	21 0	22 4	68	71	78	80	79	85	Satisfactory
17	6 7	550	20	370	670	650	720	690	1000	20	28	20	25	19	29	7 6	9 0	5 8	6 1	4 7	5 8	22 3	23 5	20 7	21 0	19 9	20 8	62	66	71	72	72	76	Satisfactory
23	6 0-6 8	400	17	310	570	570	780	750	800	17	20	17	20	16	18	7 8	10 6	5 7	8 6	5.5	6 6	21 9	23 4	20 4	21 7	19 0	20 6	55	65	60	73	67	73	Satisfactory
7	8 5	150	7	200	230	380	490	500	520	16	18	19	20	18	26	6 1	7 5	5 0	5 4	4 0	4 8	24 1	25 4	23 4	23 7	22 6	23 3	71	75	77	79	79	82	Border plastic
8	7 0	350	13	220	360	560	680	670	840	17	23	18	22	17	22	8 5	10 1	5 8	6 8	5 1	6 3	23 5	24 8	21 2	22 1	20 6	21 6	59	64	69	73	71	75	Border plastic
9	7 2	550	19	340	620	800	950	760	1150	17	22	15	24	15	26	5 2	9 5	4 2	5 2	3 2	4 5	21 2	24 5	20 4	21 2	19 5	20 6	60	75	75	79	78	84	Border plastic
17	7 5	550	20	470	570	600	800	700	780	22	24	25	29	23	28	4 6	6 8	3 5	4 6	2 5	4 1	21 4	23 2	20 5	21 4	19 7	21 0	71	79	79	83	81	87	Border plastic
7	9 5-10 6	150	7	130	240	240	410	300	520	19	25	21	29	21	31	5 4	6 2	3 2	3 9	1 5	3 2	25 6	28 2	24 3	26 5	23 3	25 5	77	79	84	88	87	94	Plastic
8	7 9-10 0	350	13	140	420	220	720	420	900	20	37	20	34	19	39	5 3	9 0	3 0	5 6	2 2	4 2	22 9	28 0	21 7	26 3	20 5	24 7	68	79	78	87	81	90	Plastic
9	8 0	550	19	450	540	630	720	640	730	23	26	27	31	28	33	4 6	5 1	3 5	4 1	2 3	3 4	22 3	22 7	21 4	21 9	20 4	21 3	78	79	81	84	84	89	Plastic
16	9 0	350	14	180	220	230	460	290	820	24	31	27	31	29	30	5 6	7 5	4 3	5 6	3 1	4 8	25 1	26 6	24 0	25 1	23 1	24 4	72	78	78	82	80	87	Plastic
22	7 5-8 3	650	24	350	730	460	780	540	720	19	29	27	42	30	48	3 9	8 6	3 3	4 7	2 2	3 9	20 8	26 1	20 3	22 6	20 0	21 9	67	81	78	85	81	90	Plastic
Comparison by Filler Content																																		
8	5 0	350	Med-13	130[b]		490[b]		1140[b]		21[b]		18[b]		18[b]		16 4[b]		12 4[b]		11 5[b]		26 5[b]		23 0[b]		22 2[b]		38[b]		46[b]		48[b]		Approaching brittle
8-16-23	6 0-8 0	350-400	Med-13-17	150	570	400	780	450	1200	17	24	17	25	16	27	7 1	13 8	4 4	10 7	3 5	9 4	21 9	26 1	20 4	23 4	19 0	22 4	47	71	54	80	58	85	Satisfactory
9-17	6 4-6 7	550	High-19-20	350	670	650	1040	690	1120	18	29	16	25	15	29	7 0	11 8	5 1	8 3	4 0	6 0	21 1	25 2	19 5	22 2	18 6	20 8	53	67	63	74	70	78	Satisfactory
7	8 5-10 6	150	Low-7	130	240	240	490	300	520	16	25	19	29	18	31	5 4	7 5	3 2	5 4	1 5	4 8	24 1	28 2	23 4	26 5	22 6	25 5	71	79	77	88	79	94	Plastic[c]
8-16	7 0-10 0	350	Med-13-14	140	420	220	720	290	900	17	37	18	34	17	39	5 3	10 1	3 0	6 8	2 2	6 3	22 9	28 0	21 2	26 3	20 5	24 7	59	79	69	87	71	90	Plastic[c]
9-17-22	7 2-8 3	550-650	High-19-24	340	730	460	950	540	1150	17	29	15	42	15	48	3 9	9 5	3 3	5 2	2 2	4 5	20 8	26 1	20 3	22 6	19 5	21 9	60	81	75	85	78	90	Plastic[c]

[a]This is a sample page from Table 7 for illustrative purposes
[b]Values are neither maximum or minimum
[c]Items classified as "Border Plastic" are included with those classified as "Plastic"

trends. However, the sand asphalt pavements in the straight tracks were all classified as plastic or border plastic and it was necessary to utilize the turnaround data to cover the range from brittle to plastic.

Asphaltic concrete.

Flow value - As shown in Table 6, there is no great difference in the flow values between the brittle and the satisfactory mixes, but at 500 and 1500 coverages all the plastic mixes have flow values higher than 20. This fact indicates that the flow value should be used as an upper limit only; that is, to indicate a trend towards too plastic a mix.

Percent voids in total mix - An inspection of the summary, Table 6, for asphaltic concrete shows that at from 500 to 1500 coverages the values of percent voids in the total mix ranged from 4.6 to 6.6 for mixes classed as approaching brittle with only one value less than 5.3. For satisfactory mixes, the maximum value was 5.3 and all other values were less than 5 percent. A division between satisfactory and approaching brittle mixes for the asphaltic concrete is fairly well-defined at about 5 percent voids in the total mix. The division between satisfactory and plastic mixes is not so well-defined and there are overlapping values. The absolute minimum value for satisfactory mixes was 2.1 percent, but, in general, the minimum values are around 3 percent or greater. For plastic mixes, the maximum value is 3.7 percent, but the average is slightly under 3 percent. The maximum values for border plastic mixes at 1500 coverages which are considered significant values are 3.3 and 3.4 percent. It appears that the boundary between plastic and satisfactory mixes for these tests is, in general, between 3 and 3.5 percent. To permit as much asphalt as possible and to assure maximum durability, a value of 3 percent is recommended as the minimum limit of percent voids total mix for satisfactory mixes.

Percent voids filled with asphalt - As shown in the summary, Table 6 values at 500 and 1500 coverages for the percent voids filled with asphalt were 70 or less for all mixes classed as approaching brittle. For satisfactory mixes, the minimum values range from 68 to 79 with only one value less than 73. It appears that the division between satisfactory and brittle mixes is between 70 and 75 and to be conservative, a value of 75 is used. In the satisfactory mixes, the maximum values range from 74 to 88. In the plastic mixes, the percent voids filled with asphalt was generally above 80 with the bulk of the values higher than 83. The division between satisfactory and unsatisfactory mixes is not well-defined, but it appears that the boundary is between 80 and 85 percent. It is recognized that the traffic in these tests was severe and also that a maximum amount of asphalt is desireable from the standpoint of durability. Therefore, in order to obtain the maximum permissible amount of asphalt, a value of 85 percent is recommended for the maximum limit of voids filled with asphalt for satisfactory mixes.

Limiting values - Based on these observed trends, limiting values for the test properties are assigned as follows:

Test Property	Brittle	Satisfactory	Plastic
Flow value	No lower limit	20 or less	More than 20
Percent voids total mix	More than 5	5 to 3	Less than 3
Percent voids filled with asphalt	Less than 75	75 to 85	More than 85

Sand asphalt - The trends indicated by the summarized data for the sand asphalt at 500 and 1500 coverages (Table 7) are similar but not identical to those found for the asphaltic concrete. In general, a flow value of 22 appears to represent the boundary between satisfactory and border plastic mixes; however, a flow of 20 is recommended as a rounded-off value. The boundary between approaching brittle and satisfactory mixes appeared to be around 7 or 8 percent voids total mix and between 60 and 65 percent voids filled with asphalt. To be conservative, values of 7 and 65 percent are recommended. The division between satisfactory and plastic mixes appears to fall at about 5 percent voids total mix and about 75 percent voids filled with asphalt. It is noted that the border plastic mixes have values for percent voids total mix slightly less than 5 percent and values for percent voids filled with asphalt that are generally a little above 75 percent. The data from the border plastic mixes are considered significant in establishing these limits. The recommended limits are summarized in the following table.

Test Property	Brittle	Satisfactory	Plastic
Flow value	No lower limit	20 or less	More than 20
Percent voids total mix	More than 7	7 to 5	Less than 5
Percent voids filled with asphalt	Less than 65	65 to 75	More than 75

Binder course mixes - The same procedure was followed for the binder course mixes. The indications obtained were that the criteria for asphaltic concrete also applied fairly well to the binder course mixes; however, it is believed that wider limits could be permitted for binder courses, particularly toward the brittle side.

CRITERIA FOR SELECTING OPTIMUM ASPHALT CONTENT

The following paragraphs present a study made to determine which test property or which combination of test properties is most applicable for selecting an optimum asphalt content. This is done by comparing the asphalt content selected by the test properties with the asphalt content determined as satisfactory on the basis of the field behavior. The average limiting value of flow, percent voids total mix, and percent voids filled with asphalt are used for these test properties. For the stability, unit weight total mix, unit weight aggregate only, and percent voids aggregate only the peak (or minimum point) in the curve of test property versus asphalt content is used. Table 8 shows the asphalt content read from the field curves of test property versus asphalt content for Mixes 7 through 15. The asphalt contents are those which correspond to the specific test properties listed on the table (peak of stability curve, etc.). The table is arranged to show a ready comparison of the average asphalt content for different groupings of the test properties. At the bottom of the table is shown the range of acceptable values based on field behavior.

From a study of the table it is seen that there is no trend for the asphalt content to vary with wheel load, which corroborates other analyses made of these data. Also, the asphalt content at 500 coverages is only slightly higher than at 1500 coverages, which indicates that the optimum asphalt content selected by these criteria will be suitable for a considerably greater number of coverages.

The values for asphalt content selected on the basis of a flow of 20, percent voids total mix of 4 percent (6.0 for sand asphalt), and percent voids filled with asphalt of 80 percent (70 percent for sand asphalt) are in good agreement with the asphalt contents determined as satisfactory under traffic, but are generally on the low end of the acceptable range,

TABLE 8[a]

ASPHALT CONTENTS SELECTED BY TEST PROPERTY CRITERIA

														Asphaltic Concrete																							
		Sand Asphalt												Coarse Aggregate - Crushed Limestone												Coarse Aggregate - Uncrushed Gravel											
		Mix 7				Mix 8				Mix 9				Mix 10				Mix 11				Mix 12				Mix 13				Mix 14				Mix 15			
Criteria Used in Selection of Asphalt Contents	Coverages	15,000-Lb Lane	37,000-Lb Lane	60,000-Lb Lane	Average	15,000-Lb Lane	37,000-Lb Lane	60,000-Lb Lane	Average	15,000-Lb Lane	37,000-Lb Lane	60,000-Lb Lane	Average	15,000-Lb Lane	37,000-Lb Lane	60,000-Lb Lane	Average	15,000-Lb Lane	37,000-Lb Lane	60,000-Lb Lane	Average	15,000-Lb Lane	37,000-Lb Lane	60,000-Lb Lane	Average	15,000-Lb Lane	37,000-Lb Lane	60,000-Lb Lane	Average	15,000-Lb Lane	37,000-Lb Lane	60,000-Lb Lane	Average	15,000-Lb Lane	37,000-Lb Lane	60,000-Lb Lane	Average
Maximum Stability	500	8 5	8 5	8 5	8 5	7 0	7 4	7 0	7 1	6 4	6 4	6 4	6 4	7 0	6 7	6 7	6 8	5.6	5 4	5 4	5 5	5 1	4.8	5 3	5 1	6 2	6 8	6 8	6 6	6 2	5 7	6 4	6 1	5 0	5 0	5 0	5 0
	1,500	8 5	8 5	9 1	8 7	7 0	7 6	7 6	7 4	6 4	6 4	6 4	6 4	6 4	6 6	7 0	6 6	5 3	5 4	5 4	5 4	4 6	4 8	4 9	4 8	5 3	6 1	6 8	6 1	6 1	5 3	6 0	5 8	5 0	5 5	5 0	5 2
Percent Voids Total Mix Asphaltic Concrete = 4 0 Sand Asphalt = 6 0	500	8 3	8 0	7 9	8 1	7 0	7 1	7 1	7 1	5 9	5 9	5 8	5 9	6 7	6 6	6 4	6 6	5 8	5 7	5 5	5 7	5 4	5 4	5 2	5 3	6 0	6 1	6 1	6 1	5 7	5 6	5 6	5 6	5 0	5 0	5 0	5 0
	1,500	8 1	8 0	7 7	7 9	6 8	6 8	6 8	6 8	5 6	5 5	5 8	5 6	6 1	6 2	6 2	6 2	5 5	5 3	5 4	5 4	4.9	5 0	5 0	5 0	5 7	5 6	5 7	5 7	5 3	5 3	5 3	5 3	4 8	4 8	4 8	4 8
Percent Voids Filled Asphaltic Concrete = 80 Sand Asphalt = 70	500	7 9	7 8	7 6	7 8	6 8	6 9	6 9	6 9	6 1	6 0	6 0	6 0	7 0	6 7	6 5	6 7	6 4	6 0	5 9	6 1	6 3	5 7	5 7	5 9	6 3	6 3	6 3	6 3	6 1	5 9	5 9	6 0	5 7	5 9	5 8	5 8
	1,500	7 8	7 8	7 4	7 7	6 7	6 7	6 8	6 7	5 8	5 6	5 8	5 7	6 3	6 3	6 3	6 3	6 0	5 6	5 7	5 8	5 6	5 3	5 4	5 4	6 1	5 9	6 1	6 0	5 8	5 7	5 6	5 7	5 4	5 6	5 3	5 4
Average of Above	500	8 2	8 1	8 0	8 1	6 9	7 1	7 0	7 0	6 1	6 1	6 1	6 1	6 9	6 7	6 5	6 7	5 9	5 7	5 6	5 8	5 6	5 3	5 4	5 4	6 2	6 4	6 4	6 3	6 0	5 7	6 0	5 9	5 2	5 3	5 3	5 3
	1,500	8 1	8 1	8 1	8.1	6 8	7 0	7 1	7 0	5 9	5 8	6 0	5 9	6 3	6 4	6 5	6 4	5 6	5 4	5 5	5 5	5 0	5 0	5 1	5 1	5 7	5 9	6 2	5 9	5 7	5 5	5 6	5 6	5 1	5 3	5 0	5 1
Maximum Unit Weight - Total Mix	500	9 2	8 7	8 9	8 9	7 6	7 9	7 7	7 7	6 7	6 6	6 4	6 6	6 3	7 0	6 3	6 5	5 6	6 2	6 0	5 9	5 1	5 8	5 4	5 4	6 5	6 8	6 7	6 7	6 0	5 9	6 2	6 1	5 2	5 2	5 1	5 2
	1,500	9 1	9 5	8.7	9 1	7 6	7 9	7 9	7 8	6 4	6 4	6 8	6 5	6 3	6 6	6 4	6 4	5 5	6 0	6 0	5 8	4 8	5 4	5 1	5 1	6 2	6 4	6 8	6 5	5 5	6 0	5 7	5 7	5 0	4 8	5 5	5 1
Average of Above	500	8 5	8 3	8 2	8 3	7 1	7 3	7 2	7 2	6 3	6 2	6 2	6 2	6 8	6 8	6 5	6 7	5 9	5 8	5 7	5 8	5 5	5 4	5 4	5 4	6 3	6 5	6 5	6 4	6 0	5 8	6 0	6 0	5 2	5 3	5 2	5 2
	1,500	8 4	8 5	8 2	8 4	7 0	7 3	7 3	7 2	6 1	6 0	6 2	6 1	6 3	6 4	6 5	6 4	5 6	5 6	5 6	5 6	5 0	5 1	5 1	5 1	5 8	6 0	6 4	6 1	5 7	5 6	5 7	5 6	5 1	5 2	5 2	5 1
Maximum Unit Weight - Aggregate Only	500	8 3	7 8	8 1	8 1	7 3	7 3	7 1	7 2	6 1	5 9	5 7	5 9	6 3	6 1	6 3	6 2	5 2	5 3	5 2	5 2	4 6	4 8	4 8	4.7	5 6	5 7	6 0	5 8	5 0	5 0	5 4	5 1	4 9	5 0	5 0	5 0
	1,500	8 3	8 4	7 6	8 1	7 2	7 5	7 6	7 4	5 8	5 9	5 9	5 9	6 3	6 2	6 3	6 3	4 8	5 1	5 3	5 1	4 0	4 8	4 5	4 4	5 2	5 5	6 0	5 6	4 9	5 0	5 0	5 0	4 7	4 2	5 0	4 6
Maximum Voids - Aggregate Only	500	8 3	7 8	8 1	8 1	7 3	7 3	7 1	7 2	6 1	5 9	5 7	5 9	6 2	6 1	6 4	6 2	5 1	5 3	5 4	5 3	4 6	5 0	4 8	4 8	5 6	5 8	6 0	5 8	5 3	5 3	5 5	5 4	5 0	4 9	5 0	4 9
	1,500	8 3	8 4	8 4	8 4	7 1	7 5	7 6	7 4	5 8	5 9	5 9	5 9	6 3	6 2	6 2	6 2	4 6	5 0	5 4	5 1	3 7	4 8	4 5	4 3	5 0	5 5	6 0	5 5	4 6	5 0	5 0	4 9	4 7	4 4	5 0	4 7
Average of Above	500	8 4	8 1	8 2	8 2	7 2	7 3	7 2	7 2	6 2	6 1	6 0	6 1	6 6	6 5	6 4	6 5	5 6	5 6	5 6	5 6	5 2	5 3	5 2	5 2	6 0	6 3	6 3	6 2	5 7	5 6	5 8	5 7	5 1	5 2	5 2	5 2
	1,500	8 4	8 4	8 2	8 3	7 1	7 3	7 4	7 3	6 0	6 0	6 1	6 0	6 3	6 4	6 4	6 3	5 3	5 4	5 5	5 4	4 7	5 0	4 9	4.8	5 6	5 8	6 2	5 9	5 4	5 4	5 4	5 4	4 9	4 9	5 1	5 0
Flow Value of 20	500	9 3	8 5	9 1	9 0	7 0	7 0	7 1	7 0	6 6	6 9	6 7	6 7	7 4	7 9	7 3	7 5	6 4	6 6	6 3	6 4	5 9	6 1	5 9	6 0	7 7	7 4	7 4	7 5	6 5	6 7	6 6	6 6	5 5	6 2	5 9	5 9
	1,500	9 1	9 3	8 7	9 0	7 0	7 0	7 0	7 0	6 6	6 4	6 6	6 5	7 3	6 7	7 5	7 2	6 1	5 8	6 1	6 0	5 6	5 2	5 6	5 5	7 4	6 6	7 0	7 0	6 3	6 0	6 2	6 2	5 6	5 2	5 5	5 4
Average of All	500	8 5	8 2	8 3	8 3	7 1	7 3	7 1	7 2	6 3	6 2	6 1	6 2	6 7	6 7	6 5	6 6	5 7	5 8	5 7	5 7	5 3	5 4	5 3	5 3	6 3	6 4	6 5	6 4	5 8	5 7	5 9	5 8	5 2	5 3	5 3	5 3
	1,500	8 5	8 6	8 2	8 4	7 1	7 3	7 3	7 2	6 1	6 0	6 2	6 1	6 4	6 4	6 6	6 5	5 4	5 5	5 6	5 5	4 8	5 0	5 0	4.9	5 8	5 9	6 3	6 1	5 5	5 5	5 5	5 5	5 0	4.9	5 2	5 0
Asphalt Contents selected by performance under traffic		7 6-8 5				6 3-7 0				6 4-7 2				6 3-7 0				5 4-6 0				4 8-5 4				6 0-6 8				5 7-6 4				5 0-5 5			

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[a] This is a sample page from Table 8 for illustrative purposes

particularly for asphaltic concrete. Inclusion of the asphalt contents selected at the peak of the stability and unit weight total mix curves gives average asphalt contents that are in better agreement with the satisfactory values than the preceding average value. The inclusion of the nearly identical values obtained from the peak of the curve for unit weight aggregate only and the minimum point on the curve for percent voids aggregate only tends to reduce the over-all average.

On the basis of this study it was concluded that the asphalt content which best fitted the field conditions was the average of the asphalt contents at the peaks of the stability curve and unit weight curves and at 4.0 percent voids total mix (6.0 percent for sand asphalt) and at 80 percent voids filled with asphalt (70 percent for sand asphalt). A secondary criterion was that the asphalt content should not be high enough to give a flow value in excess of 20. These values were selected to give an optimum asphalt content as near the upper range of the acceptable values as possible. An attempt was made to develop one set of criteria for both asphaltic concrete and sand asphalts, but all compromise criteria resulted in a reduction in asphalt content for the asphaltic concrete and an increase in the asphalt content for the sand asphalt, which were undesirable. It should be noted that the voids in the total mix and the percent total voids filled with asphalt are derived from the specific gravities of the aggregates. Any error in the determination of these gravities might lead to serious erroneous conclusions. The inclusion of values obtained by a simple weight per cu. ft. determination plus measurement of stability and flow is considered desirable to avoid erroneous assumptions or determination of specific gravities from unrepresentative samples.

It is thus seen that criteria were established for selecting an optimum asphalt content for asphalt paving mixtures. In summation, these criteria are repeated below in tabular form.

Test Property	Basis for Selecting Asphalt Content	
	Asphaltic Concrete	Sand Asphalt
Flow	Not applicable*	Not applicable*
Stability	Peak of curve	Peak of curve
Unit weight total mix	Peak of curve	Peak of curve
Percent voids total mix	4 percent	6 percent
Percent voids filled with asphalt	80 percent	70 percent

*Must not be above 20.

STABILITY AND THICKNESS REQUIREMENTS

In addition to the analysis of criteria for evaluating mixtures and selecting optimum asphalt content, analyses were also made to determine minimum stability and thickness requirements for pavements placed over high, medium, and low quality bases. The basic data and detailed analyses are not presented. The results, however, are summarized as follows:

Stability - In sections 1, 2, and 3, where the base was high quality, it was found that stability alone was not a satisfactory criterion for evaluating the ability of a pavement to resist displacement under repetitive wheel loads. A reason for this is that the curve of stability versus asphalt content peaks, making it possible to obtain equal stability values in any mix, one on the rich side and one on the lean side. The mix on the rich side will displace under traffic, whereas the one on the lean side will not. Using the criteria of actual behavior under traffic to eliminate those mixes on the rich side, an analysis was made to determine the effect of variations in stability value for those mixes that did not displace under traffic. This

analysis showed that the minimum stability values that were tested were satisfactory and there was no apparent advantage in obtaining higher stability mixes. On this basis, it was not possible to determine if mixes composed of widely different physical properties are equal if the stability values are equal. In this test, satisfactory mixes did not necessarily have the same stability. A minimum stability requirement of 500 lbs. was tentatively established, although the performance of the test section under purely rolling traffic indicated that somewhat lower values would have been satisfactory. The selection of 500 as a minimum design requirement has merit, since it can be obtained easily and economically and will provide a safety factor for pavements subjected to the impact and twisting action of airplane operations. Also requiring this minimum value will insure better gradation, closer control, and a mix that may be laid and rolled satisfactorily. The minimum value was generally substantiated by the data from the turnaround areas where the base was of reasonably high quality.

In the evaluation of pavements on the sand-clay-loess base there was no trend for high-stability mixes to be more effective in preventing shear deformation in the underlying base; however, on the sand-loess base there was a trend for the high-stability mixes to be more effective in both asphaltic concrete and sand asphalt mixes. This trend was only general, and, because of inconsistencies in individual results, specific ratings of given stability values could not be made. Also, the beneficial effect of a high-stability mix was not as pronounced as the beneficial effect of increasing the pavement thickness. For this reason no attempt was made to assign variable thickness requirements depending on the stability of the mix.

Thickness - Evaluations of the thicknesses required to prevent shear deformation in the underlying base were made for each section, which assumes that the thickness will be composed of a good-quality mix, i.e., one which conforms to the recommended design criteria outlined in preceding paragraphs. The following table summarizes the evaluation.

It should be noted that 1½ in. is shown as adequate for the 15,000-lb. wheel load on a 60 CBR base. Actually, no pavement thinner than 2 in. was placed on this base. However, since 1½ in. thickness was found to be satisfactory for both a 50 and an 80 CBR base, the assumption that it would also be satisfactory for the bracketed case is reasonable.

A study of the variation of settlement with thickness shows that, in general, the settlement was less under the thicker pavements. The following table shows the thickness required to limit the settlement to less than 1 in. (Table, top of page 61.)

Base	Base CBR Percent	Thickness Requirements to Prevent Shear in Underlying Base - Inches: 15,000- lb Wheel Load	37,000- and 60,000- lb. Wheel Loads
Sand-loess	40	3 - 5	3 - 5
Sand-clay-loess	50	1 - ½	3 - 5
Gravel	60	1 - ½	2
Crushed limestone	80	1 - ½	1 - ½

It is noted that for the 37,000- and 60,000-lb. loads a 3-in. pavement limited the settlement to less than 1 in. on the crushed limestone base, but for the other two bases it was necessary to go to the 5-in. pavement to achieve the desired reduction. For the 15,000-lb. wheel load, there was no advantage gained in going to the thicker pavements for any base.

The following table combines the data for thickness requirements to prevent shear deformation in the base with those required to prevent pronounced consolidation. (Table, center of page 61.)

Base	Base CBR Percent	Thickness Requirements to Limit Settlement to Less Than 1 Inch - Inches: 15,000- lb Wheel Load	37,000- and 60,000-lb Wheel Loads
Sand-loess	40	1.5	5
Sand-clay-loess	50	1.5	5
Crushed limestone	80	1.5	3

The pavement thicknesses recommended in the table above are based on an asphaltic concrete of good quality conforming to the limiting criteria given in preceding paragraphs. For bases with CBR values in excess of 60 where there is no problem of preventing shear deformation in the base, sand asphalt can be substituted for the asphaltic concrete on an inch for inch basis. The thickness of good quality sand asphalt (conforming to the criteria given in preceding paragraphs) required to prevent shear deformation in bases with CBR values of 40 and 50 was not determined in these tests, and to be conservative asphaltic concrete is recommended for these bases. The values recommended in the table above represent thicknesses that are considered reasonable even though in some cases they are in excess of the values indicated by the data. A minimum thickness of 2 in. is recommended for the 15,000-lb. wheel load (or for lighter loads) because a construction tolerance is believed desirable. It also should be noted that 3 in. is recommended for the 15,000-lb. wheel load on the 50 CBR base. This was done because a differential on a sliding scale for bases with CBR between 40 and 60 is reasonable. The selection of a 4-in. thickness for 37,000- and 60,000-lb. wheel loads on a CBR of 60 also was based on obtaining a differential on a sliding scale for bases with CBR values between 50 and 80.

Summary of Thickness Requirements

CBR of Base	15,000-Lb Wheel Load: Shear	Consolidation	Recommended Thickness	37,000- and 60,000-Lb Wheel Loads: Shear	Consolidation	Recommended Thickness
40	3 - 5	1.5	4	3 - 5	5	5
50	1.5	1.5	3	3 - 5	5	5
60	1.5	-	2	2	-	4
80	1.5	1.5	2	1.5	3	3

EFFECT OF AGGREGATE TYPE, GRADATION, AND FILLER

In addition to the determination of limiting design criteria and thickness requirements, analyses were made of the effect of aggregate type and gradation and of the effect of quantity of filler. The results are summarized as follows:

Aggregate type - Where the underlying base was of high quality, properly proportioned asphaltic concrete pavements containing either crushed limestone or uncrushed gravel performed satisfactorily. The sand asphalt pavements placed on high quality base in the main test track showed more tire printing and slightly more rutting and shoving and were, in general, not the equal of the asphaltic concretes. However, this comparison may not be strictly valid since almost all the sand asphalt pavements contained an excess of asphalt cement and were composed of the same mixture for the full thickness, whereas comparable asphaltic concretes included mixtures with a satisfactory asphalt content which were underlain by binder courses except in the 1½-in. pavements. In the turnarounds the performance of sand asphalt pavements with lower asphalt contents was satisfactory and comparable to the asphaltic concrete items. In the main test track where the underlying base was of poor quality, the asphaltic concrete pavements containing crushed limestone were definitely super-

ior to the sand asphalt items. None of the asphaltic concrete mixes containing uncrushed gravel were placed on the low quality bases. The best sand asphalt pavement was not quite satisfactory in preventing shear deformation in the inferior base, even at a thickness of 6 in. Three inches of good quality asphaltic concrete were generally satisfactory and 5 in. were always adequate. However, the comparison may not be strictly valid, since the sand asphalt mixes were rich in asphalt as previously discussed.

Aggregate gradation - A comparison of the performance of pavements consisting of both sand asphalt and asphaltic concrete in which the aggregates were poorly graded with corresponding pavements containing well-graded aggregates revealed no significant differences in behavior when subjected to traffic of the wheel loads used in the test. However, the data are too limited in scope to determine whether all types of poorly graded mixtures would perform in a satisfactory manner under the test conditions.

Filler - Differences in stability on the main test track were obtained by placing asphalt mixtures in items at low, medium, and high percentages of filler. For asphaltic concrete, the adjective low denotes no filler added to the mixture; medium, a total of 5 to 7 percent filler; and high, about 10 percent filler in the aggregate mixture. In the turnarounds a few items were placed at filler contents as high as 18 percent. For sand asphalt the same range for low, medium, and high is interpreted to mean: low, none added; medium, about 13 percent; and high. about 19 percent. In the turnarounds a few items were placed with filler content as high as 24 percent. Where the underlying base was of high quality, the test data reveal little significant difference in mixtures which were comparable except for filler content. During warm weather traffic, the extremely high filler mixes on the turnarounds showed slightly more cracking and other detrimental behavior than those with normal filler contents. During cold weather traffic, the high filler mixes on the main test tracks showed more aggregate shelling but no more cracking than those with lower filler contents. Where the underlying base was of low quality, the mixes containing a high percentage of filler gave, in general, more satisfactory performances than those with low filler; however, the results were inconclusive. Certain test properties such as percent voids total mix and percent voids filled with asphalt were approximately the same at optimum asphalt regardless of filler content. This fact indicates that both filler and asphalt cement function as void filling materials and within limits can be used to supplement each other.

CONCLUSIONS

The following conclusions are extracted directly from Appendix D of Waterways Experiment Station TM 3-254 *(1)*. It should be noted that some of the conclusions are not supported by data and discussions in this paper. Complete data are given in TM 3-254 *(1)*.

Effect of wheel load - A comparison of the values for the various test properties in mixtures which were identical but subjected to different wheel loads shows that, at equal coverages, the three wheel loads produced approximately the same test properties. However, the 15,000-lb. wheel load was much less severe in producing shear deformation and consolidation in the underlying base than the 37,000-lb. single or the 60,000-lb. dual wheel load. The two heavier wheel loads were approximately equal in this respect.

Effect of variation in physical properties - The most important variable in an asphalt paving mixture is the quantity of asphalt. Mixtures having a wide range of physical properties performed satisfactorily where the asphalt content was properly proportioned.

In pavements placed on a high-quality base, the type of aggregate was a minor variable as asphaltic concretes containing both crushed limestone and uncrushed gravel coarse aggregate and sand asphalts performed satisfactorily when the mixture was properly proportioned.

In these tests the asphaltic concrete

containing crushed limestone aggregate was more effective in preventing shear deformation in the underlying low-quality base than the sand asphalt, but the effect of aggregate type in this respect was not fully studied. The required thickness of good-quality sand asphalt pavement (conforming to criteria given in a later conclusion) to protect a low-quality base was not determined.

These tests did not reveal any important differences between well-graded and poorly-graded aggregates. Asphaltic concrete mixtures containing poorly-graded aggregates gave performances which were reasonably comparable to those in which the grading conformed to standard practice. It is possible that in some cases poorly-graded aggregates (not including filler material) may produce satisfactory pavements.

Stability - The stability value is not a satisfactory indication of the ability of a mix to resist displacement under traffic due to the fact that equal stability values can be obtained in a given mix at different asphalt contents, one below optimum and one above optimum.

For pavements which are to be subjected to wheel loads of the magnitude used in these tests a minimum stability value of 500 lbs. is recommended.

There was no apparent benefit gained in these tests from using a mixture with a stability higher than 500 lbs. when the base had a CBR of 80 or better.

There was a general trend for mixes with higher stabilities to be more effective in preventing shear deformation in an underlying base of medium or low quality, but no specific ratings could be assigned. A higher stability was not as effective as additional thickness in preventing shear deformation.

No conclusions can be reached relative to the performance of good quality (conforming to criteria given in a later conclusion) mixes of equal stability but composed of widely different physical properties when subjected to the wheel loads used in this test.

Thickness requirements - A high-quality base does not require a pavement to protect it from shear deformation; however, for the heavier wheel loads, thicker pavements tend to reduce consolidation in the underlying materials.

Bases of inferior quality may be protected from shear deformation by increased thicknesses of asphaltic concrete provided the mixture is properly designed. In addition, the increased pavement will reduce consolidation of the underlying materials.

The following thicknesses of good quality asphaltic concrete are recommended to prevent shear deformation in the base and reduce settlement of the pavement surface to less than 1 in. For bases with CBR values of 60 or better an equal thickness of sand asphalt may be substituted.

	Thickness in Inches	
CBR	15,000-lb. Wheel load	37,000- and 60,000-lb. Wheel loads
40	4	5
50	3	5
60	2	4
80	2	3

Surface treatments - Under the conditions of this test, the surface treatments were adequate over high-quality bases, but were not adequate for inferior bases.

Marshall optimum asphalt content - The asphalt content as determined by the original Marshall test was higher than the optimum value required by traffic as a result of compaction (see also conclusions on design below).

Design of Asphalt paving mixtures - The following conclusions are presented regarding the design of asphalt paving mixtures. As stated in an earlier conclusion, the quantity of asphalt is the most important variable in an asphalt paving mixture. The following conclusions deal primarily with criteria for use in selecting the proper asphalt content, and for evaluating mixes from the standpoint of resisting displacement under traffic.

Traffic compaction increases the density of an asphalt paving mixture, and this increase in density reduces the quantity of asphalt required to fill a

given percentage of the voids.

It is necessary to design the mixture with an asphalt content that will be satisfactory under traffic in order to prevent the mix from becoming plastic due to increase in density under traffic.

The compactive effort in these tests was a function of the coverages and the tire contact pressure rather than the wheel load.

The results of these tests at between 500 and 1500 coverages are applicable for determining the laboratory compactive effort to be used in designing asphalt paving mixtures.

The percent voids total mix and the percent voids filled with asphalt are usable indexes of the capacity of the mix to resist displacement under traffic. However, they are based on the specific gravity of the component parts of the mixture and are subject to the inaccuracies that may occur in specific gravity determination. These tests indicate that mixtures should be designed so that when compacted under traffic they will have values within the following ranges. The values are considered tentative.

Type of Mix	Percent Voids Total Mix	Percent Voids Filled With Asphalt
Asphaltic Concrete	3 to 5	75 to 85
Sand Asphalt	5 to 7	65 to 75

The peaks of the curves of stability versus asphalt and unit weight total mix versus asphalt are excellent tools for selecting the proper asphalt content. These values are measurable directly and are independent of specific gravity determination.

The optimum asphalt content to assure a satisfactory mix should be the average of the asphalt contents that give the properties stated in the two conclusions directly above when the mixture is compacted to the degree stated above.

The flow value is an excellent index of the plasticity of a mix. In general, values in excess of 20 indicate a mix that will displace under traffic, and a value of 20 is tentatively set as maximum.

CORRELATIONS OF LABORATORY AND FIELD DATA

by WOODLAND G. SHOCKLEY*

INTRODUCTION

The original procedure for compacting laboratory specimens for the Marshall stability test had produced densities that were comparable to those obtained in the construction of pavements on the few airfields investigated. As has been noted in the discussion of tracking operations on the asphalt test section, it became apparent, from results of tests on cored samples, that compaction of the pavement was taking place under the traffic applied. The increased density of the pavement reduced the void space and allowed the asphalt to fill the voids more completely. In the event the pavements contain sufficient asphalt to overfill the voids when the density is increased by traffic, the pavements may flush and become unstable. Some indication of increased density had been obtained in previous laboratory tests in which the compactive effort was varied. An increase in the compactive effort resulted in higher densities and a lower optimum asphalt content. Therefore, it was decided to investigate the possibility of increasing the compactive effort in the laboratory test to obtain densities approximating those attained under traffic, such that the lower optimum asphalt content would provide a pavement design which would not have an excess of asphalt as a result of compaction at any time during the life of the pavement.

During the progress of the studies reported in this paper, a number of different compactive efforts were used in establishing the proper effort to be used in preparing laboratory specimens for the design and construction control of asphalt pavements. Inasmuch as it is rather difficult to keep in mind all of the factors involved in the compaction procedures, they are summarized in the following tabulation. Also shown are abbreviated designations by which the various procedures are called in this paper. (Table, page 66.)

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STUDIES FOR SELECTION OF LABORATORY DESIGN COMPACTION PROCEDURE

The data from the 9 principal mixes used in the test section were plotted as test property curves for several intervals of traffic coverages. Values of 500, 1000, and 1500 or 3500 coverages were selected for analysis to cover the range of traffic experienced in the test section. The major changes in the pavement properties had occurred before 500 coverages, as explained in a previous paper of this symposium. Based on a preliminary analysis of the traffic data, an optimum asphalt content was selected for each mix at each interval of coverage. The results of this study, including the criteria used for selection of optimum asphalt content, are shown on Table 1. In the laboratory the same aggregate materials corresponding to the test section mixes were prepared with a range of asphalt contents and compacted at three compactive efforts; namely, 40, 55, and 75 blows with the modified AASHO hammer. The optimum asphalt content for each compactive effort was selected using the same test properties criteria as for the traffic data; these values are also shown on Table 1. Inspection of the table shows that for a given mix there was very little difference in field optimum asphalt content at any specific coverage for all three lanes; thus, it was proper to select an average value of optimum asphalt on data from all

Compactive Effort	Terminology used in this paper
15 blows on top of specimen with Modified AASHO hammer (10-lb. weight 18-in. drop, 1.95-in. diameter foot) plus 5,000-lb. static load	Original procedure
40, 55, or 75 blows on both top and bottom of specimen with Modified AASHO hammer plus 500-lb static leveling load	40, 55, or 75 blows with Modified AASHO hammer
8, 10, 15, 20, 25, or 50 blows on both top and bottom of specimen with 12.5-lb. hammer, 18-in. drop, 3-7/8 in. diam. foot	8, 10, 15, 20, 25, or 50 blows with 12.5-lb. hammer, 3-7/8-in. foot
15, 40, or 50 blows on both top and bottom of specimen with 10-lb. hammer, 18-in. drop, 3-7/8 in. diam. foot	15, 40 or 50 blows with 10-lb. hammer, 3-7/8-in. foot

three lanes for each mix as representing the field condition. The optimum asphalt contents for specimens compacted with 55 blows were reasonably comparable to the values selected from the traffic tests for all mixes. On the basis of these comparisons, a compactive effort of 55 blows with the modified AASHO was tentatively selected as closely approximating the results obtained by traffic at from 500 to 1500 coverages of the wheel loads used in the test.

The laboratory compaction procedure tentatively selected involved considerably more work to prepare specimens for test than did the original procedure using 15 blows of the hammer. In addition, it was noted that some degradation was occurring in the compacted aggregates near the surface of the specimen. For these reasons a laboratory study was initiated in an effort to achieve the same results in compaction with less effort on the part of laboratory technicians and to eliminate the degradation of aggregates. A detailed discussion of the investigation is beyond the scope of this paper; however,

is beyond the scope of this paper; however,

the various methods tried and the final results of the study are presented.

Hammer studies were conducted using various sizes of hammer face from 1 in. up to 3-7/8 in. in diameter. Different design of hammer face, such as bullet-nose, stair stepped, etc., were investigated. The hammer weight was increased from 10 lb. to 12½ and to 15 lb. In addition, the shape of the compaction mold base was varied in an effort to produce a kneading action in preparation of the test specimens. Finally, other types of compaction were tried; namely, static compaction and "drop mold" compaction wherein the mold was weighted and dropped a number of times through a definite distance. The results of these tests indicated that the static method of compaction used was not satisfactory and that none of the dynamic methods tried would materially reduce the work required in the laboratory to prepare specimens, although there was a difference in the efficiency of the hammers with the different face diameters. A 3-in. diameter was most efficient and the 3-7/8-in. diameter was slightly less efficient than the 1.95-in. diameter of the Modified AASHO hammer. However, the large diameter hammer face (3-7/8 in.) was effective in reducing degradation of aggregates and eliminated the necessity for moving the hammer in the mold after each blow. Therefore, the following procedure was adopted: the hammer face was increased to 3-7/8 in. diameter, the weight was increased to 12½-lb.,

TABLE 1

COMPARISON OF OPTIMUM ASPHALT CONTENTS TRAFFIC AND LABORATORY COMPACTION

Asphalt Test Section

Mix No	Type	Asphalt Contents Placed on Test Section	Optimum Asphalt, Traffic Compaction: 15,000-Lb Lane 500 Coverages	15,000-Lb Lane 1000 Coverages	15,000-Lb Lane 3500 Coverages	37,000-Lb Lane 500 Coverages	37,000-Lb Lane 1000 Coverages	37,000-Lb Lane 1500 Coverages	60,000-Lb Lane 500 Coverages	60,000-Lb Lane 1000 Coverages	60,000-Lb Lane 1500 Coverages	Average	Optimum Asphalt, Laboratory Compaction: Modified AASHO Hammer 40 Blows	Modified AASHO Hammer 55 Blows	Modified AASHO Hammer 75 Blows	12-½-Lb Hammer 3-7/8-In Foot 50 Blows
7	Sand Asphalt 7% Filler	8 5, 9 5, 10 6	8 6	8 5	8 5	8 4	8 4	8 5	8 5	8 6	8.4	8.5	8 5	8 3	8 1	8 3
8	Sand Asphalt, 13% Filler	7 0, 7 9, 8 8	6 9	6 8	6 9	7 3	7 5	7 5	7 2	7 2	7.5	7 2	7 2	6 8	6 7	6 9
9	Sand Asphalt, 19% Filler	6 4, 7 2, 8 0	6 3	6 0	6 3	6 5	6.3	6 2	6 3	6 3	6.3	6 3	6.5	6 4	5 9	6 3
10	Asphaltic Concrete, Crushed Limestone, 2% Filler	5 6, 6 3, 7 0	6 3	6 1	6 1	6 5	6.4	6 3	6 4	6 2	6.3	6 3	6.7	6 7	6 3	6 3
11	Asphaltic Concrete, Crushed Limestone, 5% Filler	4 8, 5 4, 6 0	5 2	5 2	4 9	5 4	5 5	5 3	5 5	5 5	5 5	5 3	5 5	5 3	5 0	5 3
12	Asphaltic Concrete, Crushed Limestone, 8% Filler	4 8, 5 4, 6 0	5 0	4 9	5 0	5 0	5 0	5 0	4 9	4 6	4 9	4 9	5 1	4 8	4 5	4 6
13	Asphaltic Concrete, Uncrushed Gravel, 4% Filler	6 0, 6 8, 7 5	6 1	6 0	6 0	6 2	6 1	6 1	6.2	6 2	6.2	6 1	6 1	6.0	6 0	6 0
14	Asphaltic Concrete, Uncrushed Gravel, 5% Filler	5 7, 6 4, 7 1	5 6	5 6	5 7	5 5	5 5	5 3	5 8	5 6	5.7	5 6	5 5	5 5	5 3	5 4
15	Asphaltic Concrete, Uncrushed Gravel, 9% Filler	4 5, 5 0, 5 5	5 0	5 0	4 7	5.0	5 0	4.8	5 0	5 0	5.0	4 9	4.8	4 9	4 9	4 4

NOTES

1 The criteria for determining optimum asphalt content were as follows

Property	Limit
Flow	15
Stability	Minimum
Unit Weight-Total Mix	Maximum
Unit Weight-Aggregate Only	Maximum
Percent Voids-Aggregate Only	Minimum
Percent Voids-Total Mix	5
Percent Voids Filled with Asphalt	75

2 A 500-lb static leveling load was used in addition to the hammer compaction

and the number of blows on each side of the specimen reduced to 50. The large hammer face also eliminated the necessity for a static leveling load on the specimen. The revised compaction method duplicated the test results obtained with the previously-used 55 blows of the Modified AASHO hammer.

The foregoing revised procedure was used in the laboratory until the results of the final detailed analysis of the traffic data from the test section became available. This detailed analysis indicated that the optimum asphalt content selected by the revised compaction method just described was on the low side of the acceptable range of values as determined by traffic tests. In addition, it was recommended by the board of consultants that the compaction hammer weight be changed back to 10 lb. in order that it would more closely correspond to the Modified AASHO hammer. These changes necessitated further studies in the laboratory to establish a compactive effort that would produce reasonable agreement between field and laboratory optimum asphalt contents using the revised criteria.

To achieve this result, the data from 9 test section mixes used in the original compaction study were re-analyzed and new optimum asphalt contents were selected. These values were compared with the range of asphalt contents indicating satisfactory pavement behavior of the mixes in the test section, from which it was determined that 40 rather than 55 blows of the Modified AASHO hammer most nearly reproduced the desired optimum asphalt contents in the laboratory.

In order to correlate the compactive effort of 40 blows of the Modified AASHO hammer with the 10-lb. hammer, 3-7/8 in. foot, the following laboratory tests were conducted. Four of the 9 test section mixes, plus a mixture containing slag aggregate, were synthesized in the laboratory and optimum asphalt determined for 40 blows of the Modified AASHO hammer, as well as for 40 and 50 blows of the 10-lb. hammer with 3-7/8 in. foot. The results of these studies are shown in Table 2. Based on the analysis of these data it was determined that 50 blows of the 10-lb., 3-7/8-in. hammer should be used in compacting laboratory specimens as representing the proper design procedure for airfield pavements.

FIELD AND LABORATORY COMPACTION CORRELATION STUDIES

General - At various times throughout the investigation, information was obtained on the relationship between densities of bituminous pavements as constructed and the compactive effort required to duplicate those densities in the laboratory. In addition, data were available on a few satisfactory airfield pavements which had been subjected to considerable traffic and which could be compared with the laboratory compaction results.

It has been mentioned previously that the original compaction procedure produced densities that were approximately the same as those obtained in the construction of the airfield pavements investigated. It should be mentioned at this time that in order to expedite airfield construction the use of 8-ton roller, both three wheel and tandem types, was permitted during the national emergency. The current Corps of Engineers' specifications require the use of rollers weighing at least 10 tons for compacting bituminous pavements. However, the revised compaction procedure adopted for the laboratory design of asphalt paving mixtures produced densities in laboratory specimens that were higher than those obtained in field construction. This procedure, therefore, could not be used for control of rolling operations during construction. It was desired to obtain further information from field construction projects and to make laboratory compaction studies on the bituminous materials to see what improvement in correlation could be made between the laboratory and field compaction procedures.

Airfield Investigation - The airfield investigation program conducted by the Waterways Experiment Station has provided considerable information on bituminous pavements. In general, at each airfield investigated, samples of the pavement were obtained at the center and edge of the runways and taxiways. These samples were

TABLE 2

COMPARISON OF OPTIMUM ASPHALT CONTENTS - LABORATORY COMPACTION

Mix No.	Type	Asphalt Content Selected from Traffic Behavior	Optimum Asphalt Content - Laboratory Compaction: Original Mixes, Modified AASHO Hammer, 40 Blows	Original Mixes, Modified AASHO Hammer, 55 Blows	Original Mixes, Modified AASHO Hammer, 75 Blows	Laboratory Blends, Modified AASHO Hammer, 40 Blows	Laboratory Blends, 10-Lb Hammer, 3-7/8-Inch Diameter Foot, 40 Blows	Laboratory Blends, 10-Lb Hammer, 3-7/8-Inch Diameter Foot, 50 Blows
7	Sand Asphalt, 7% Filler	7 6 - 8 5	8 5	8 2	8 0	-	-	-
8	Sand Asphalt, 13% Filler	6 3 - 7 0	7 1	6 8	6 7	7 3	7 4	7 2
9	Sand Asphalt, 19% Filler	6 4 - 7 2	6 4	6 3	6 1	-	-	-
10	Asphaltic Concrete, Crushed Limestone, 2% Filler	6 3 - 7 0	7 0	6 9	6 6	-	-	-
11	Asphaltic Concrete, Crushed Limestone, 5% Filler	5 4 - 6 0	6 0	5 8	5 5	5 6	6 0	5 7
12	Asphaltic Concrete, Crushed Limestone, 8% Filler	4 8 - 5 4	5 5	5 2	5 1	-	-	-
13	Asphaltic Concrete, Uncrushed Gravel, 4% Filler	6 0 - 6 8	6 7	6 6	6 4	-	-	-
14	Asphaltic Concrete, Uncrushed Gravel, 5% Filler	5 7 - 6 4	6 0	5 9	5 9	5 3	5 5	5 4
15	Asphaltic Concrete, Uncrushed Gravel, 9% Filler	5 0 - 5 5	5 3	5 2	5 4	-	-	-
-	Asphaltic Concrete, Slag, 8% Filler	-	-	-	-	5 3	5 6	5 3

NOTES

1 A 500-lb static leveling load was used in addition to the hammer compaction for the Modified AASHO hammer only

tested for density, stability, and flow, and the other test properties were computed. From the same locations as the cored samples, disturbed pavement samples were also obtained and tested in the laboratory. The samples were reheated and compacted with 10 blows and 50 blows of the 12.5-lb. hammer, 3-7/8 in. foot. These efforts were selected on the basis of preliminary studies that indicated the 10-blow compaction would approximate construction rolling densities, whereas the 50-blow compaction had been tentatively selected for design of asphalt pavements. It may be well to mention here that the 12.5-lb. hammer with 3-7/8 in. foot was used in these studies since they were concurrent with the analysis of the asphalt test section data and the final hammer design (10-lb., 3-7/8 in. foot) was not selected until a later date.

It has been determined from other investigations that the major portion of airplane traffic is concentrated in the center portion of runways and taxiways, whereas the edges receive very little traffic. Table 3 shows for several airfields the unit weights of pavements at the center of runways and taxiways compared with the 50-blow laboratory compaction results. It will be noted that the pavement densities at the center of runways average about 6 lb. per cu. ft. less than the 50-blow laboratory value, whereas the unit weights at the center of taxiways average about 2 lb. per cu. ft. below the 50-blow laboratory values. These data tend to show that the laboratory design compaction have considerably higher densities than were obtained on runways and slightly higher than were obtained on taxiways. This would indicate that the laboratory compaction procedures might be somewhat conservative for design purposes.

In order to determine the relationship between field as-built and laboratory compaction, a comparison was made between the unit weights at the edges of runways, assumed to approximate the as-constructed condition, and the 10 blow laboratory compaction. Table 4 shows these data. A comparison of the unit weights of the 10-blow laboratory compaction with the runway edge densities shows that in only two cases (Camp Campbell and La Junta, Pits 4 and 5) did the unit weight exceed that of the laboratory specimens. It would appear from this comparison that 10 blows of the 12.5-lb. hammer, 3-7/8 in. foot on laboratory specimens would be a little severe to use as a criterion to establish densities to be attained in construction.

TABLE 3

Comparison of Field and Laboratory Densities - Runway and Taxiway Centers-Airfield Studies

	Unit Weight -- Lb Per Cu. Ft			
	Runways		Taxiways	
Field	Center	50 Blow Laboratory	Center	50 Blow Laboratory
Lawson Field, Ga.	-	-	139	143
	-	-	138	143
Jackson AAB, Miss.	141	139	-	-
Camp Campbell, Tenn.	136	143	-	-
Berry Field, Tenn.	136	147	-	-
	143	147	-	-
Bergstrom Field, Tex.	147	148	147	148
Dodge City, Kans.	-	-	144	146
Woodward, Okla.	-	-	138	142
La Junta, Colo.	138	147	148	146
	141	148	-	-
Rocky Ford, Colo	136	142	-	-
Pueblo, Colo.	139	146	144	146
	138	145	-	-

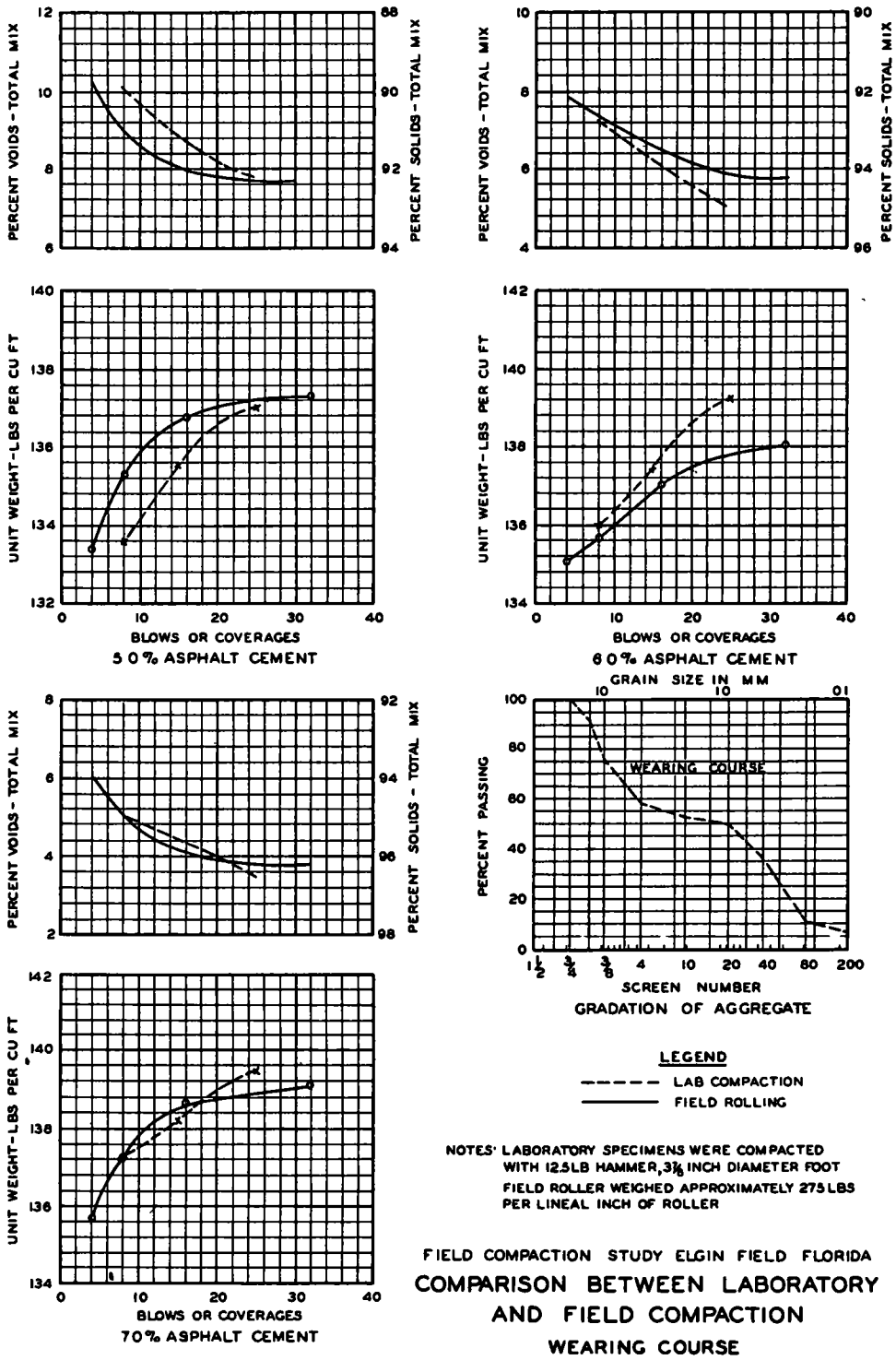
TABLE 4

Comparison of Field and Laboratory Densities Runway Edges - Airfield Studies

	Unit Weight -- Lb. Per Cu Ft	
Field	Runway Edge	Laboratory
Bergstrom Field, Tex.	142	144
Berry Field, Tenn	135	140
Camp Campbell, Tenn.	135	134
Jackson AAB, Miss.	134	140
La Junta, Colo.		
Pits 1 and 2	136	143
Pits 4 and 5	134	132
Pueblo, Colo.		
Pits 3 and 6	134	140
Pits 5 and 4	136	138
Rocky Ford, Colo.	134	137

Field Rolling Studies

Eglin Field, Florida - The Waterways Experiment Station, in conjunction with the Mobile District, Corps of Engineers, conducted a rolling study at Eglin Field, Florida, in September 1946. The pavement was an asphaltic concrete in which a 1-in. maximum size slag was used as the coarse aggregate. The fine aggregate was a local sand and the filler was limestone



FIELD COMPACTION STUDY ELGIN FIELD FLORIDA

COMPARISON BETWEEN LABORATORY AND FIELD COMPACTION

WEARING COURSE

Figure 1

dust. Ten-ton tandem rollers which produce approximately 275 lb. pressure per lineal inch of roller were used. Sections of pavement were placed at three percentages of asphalt and compacted with 4, 8, 16, and 32 passes of the rollers. Samples were cored from the completed pavements and tested. Samples were also taken of the plant-mixed asphaltic concrete for each section and specimens were compacted in the field laboratory using 8, 15, and 25 blows, each side, using the 12.5-lb. hammer, 3-7/8 in. foot. All specimens were tested for density, stability and flow.

The results of tests on the field and laboratory samples of the wearing course, together with the gradation of the pavement aggregates, are shown on Figure 1. Plotted on the figure are curves of density versus number of blows for laboratory compacted samples and density versus roller coverages for the field samples. It appears that in general the densities increase as the number of blows or coverages increase. It is assumed that 32 coverages constitutes excessive rolling and that satisfactory densities should be obtained with about 8 to 12 coverages. On this basis, a comparison of the laboratory data with field densities at 8 to 12 coverages indicated that 10 blows of the 12.5-lb. hammer, 3-7/8 in. foot produced laboratory densities that were in reasonable agreement with good field rolling.

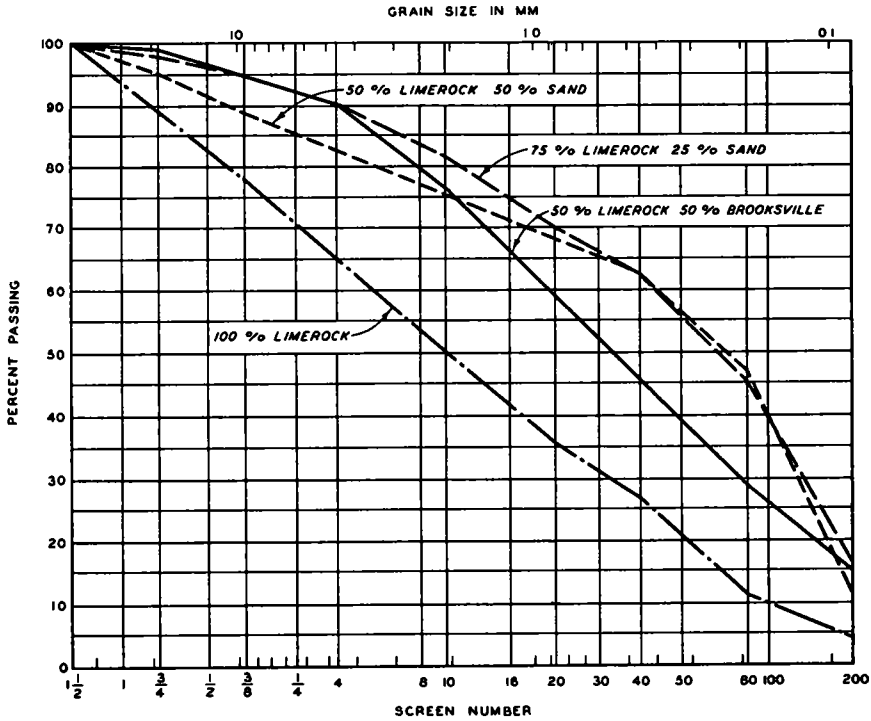
MacDill, Field, Florida. - In June 1946, the Waterways Experiment Station was invited by the Savannah District, CE, to participate in the construction of four test sections at MacDill Field, Florida. The primary purpose of the tests was to determine the suitability of Florida limerock aggregate for construction of hot-mix bituminous pavements. However, only the features pertaining to field and laboratory compaction are discussed here. The four test sections each consisted of a different type of wearing course placed at various asphalt contents on a previously prepared base. The composition of the test sections is shown on Table 5. Gradations of the aggregates are shown on Figure 2. The wearing courses were approximately 1-3/4 in. thick and were compacted with 6 straight and 4 diagonal coverages of an 8-ton roller. Samples of the completed pavement were obtained from each of the sections and test properties were determined. Samples of the plant-mixed materials for each section were reheated and compacted in the laboratory with 10, 15, and 20 blows of the 12.5-lb. hammer with 3-7/8 in. foot.

TABLE 5

Composition of Test Sections, MacDill Field, Florida

Section	Type of Aggregate	Asphalt Cement Percent
I	100 Percent Limerock	9, 10, 11, 12
II	50 Percent Limerock 50 Percent Local Sand	8, 9, 10, 11
III	75 Percent Limerock 25 Percent Local Sand	9, 10, 11, 12
IV	50 Percent Limerock 50 Percent Limestone Screenings	8, 9, 10, 11

Curves of density versus asphalt content for field and laboratory compacted samples for Section II, 50 percent limerock and 50 percent sand, are shown on Figure 3. These results may be considered comparable to those obtained on the other test sections. A comparison of the laboratory and field densities for the four test sections showed that 10 to 15 blows of



MACDILL FIELD

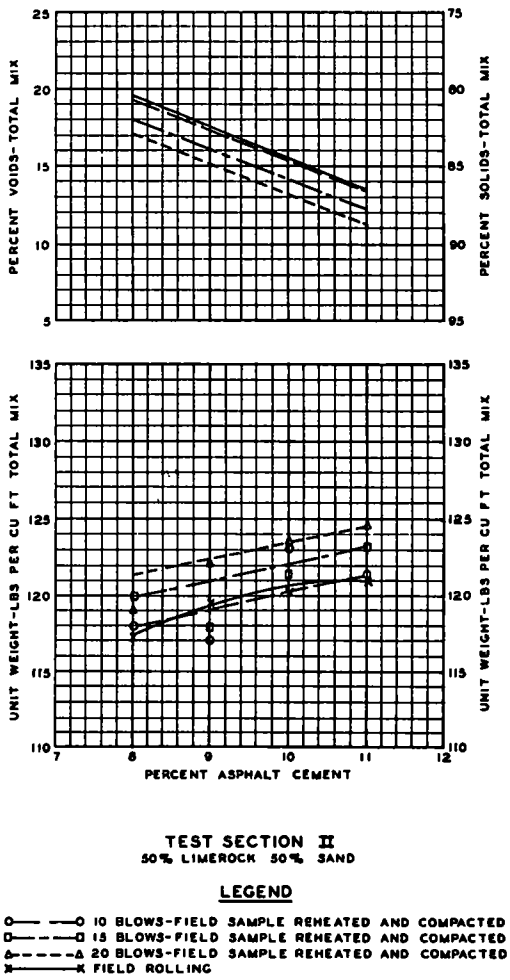
GRADATION OF PAVEMENT AGGREGATES

Figure 2

12.5-lb. hammer, 3-7/8 in. foot were required to produce densities in the laboratory that were comparable to those obtained in field construction.

Stockton Test Section No. 2 - During the construction and operation of Test Section No. 2 for very heavy wheel loads at Stockton, California, the Sacramento District, CE, furnished the Waterways Experiment Station with samples of the pavement for testing purposes. Of especial interest are the cores representing the as-constructed condition of the pavement. Six basic designs were used in constructing the wearing and binder courses for the test sections. Materials used in the bituminous mixtures were crushed granite, crushed river gravel, sand, sandy loam, and limestone dust. The pavement was placed in courses and each course was initially rolled with a 3-wheeled roller and finished with a 10-ton tandem roller. It is not known how many coverages were applied, but an effort was made to obtain high densities in construction. Cores were cut from the completed pavement outside of the traffic lanes and tested for density, stability and flow. Samples of the unmixed pavement aggregates were also furnished the Waterways Experiment Station. The six pavement designs were duplicated in the laboratory and specimens were compacted using 50 blows of the 12.5-lb. hammer, 3-7/8 in. foot. Test properties were determined for the laboratory specimens.

Table 6 shows a comparison of unit weights for the field and laboratory specimens for the six pavement designs. It may be seen from the tabulation that for the binder courses the unit weights of the field samples ranged from 7 lb. per cu.



Mac Dill Field, Florida

Figure 3. Comparison Between Laboratory and Field Compaction

ft. above to 3 lb. per cu. ft. below the corresponding laboratory densities. In the wearing courses the unit weights of the field samples ranged from equal to 6 lb. per cu. ft. below the laboratory densities. Since the laboratory compactive effort was 50 blows of the 12.5-lb. hammer, 3-7/8 in. foot, and not the lesser 10-blow effort which approximated construction densities in the other projects investigated, it is apparent that in practically all cases the pavement densities were relatively high. This may have been due to the use of 10-ton rollers on the project or, in part, to compaction by casual traffic over these areas.

Summary - Investigations at seven airfields throughout the country showed that in the majority of cases 10 blows of the 12.5-lb. hammer with 3-7/8 in. foot procuded higher laboratory densities than were obtained in the construction of the pavement, but in two cases the laboratory density was exceeded in the field. Further information was obtained at two carefully controlled rolling studies at Eglin Field and MacDill Field, Florida, which indicated that it was possible to obtain pavement densities equivalent to those obtained in the laboratory with 10 blows of the 12.5-lb. hammer, 3-7/8 in. foot. Data from Stockton No. 2 test section also indicated that high pavement densities might be obtained in construction.

The field and laboratory correlations of as-constructed pavement densities were based on the original laboratory compaction procedure and on the laboratory procedure using the 12.5-lb. hammer, 3-7/8 in. foot. Inasmuch as neither of these procedures used the 10-lb., 3-7/8 in. foot hammer finally selected for the laboratory design test, it was necessary to correlate between the various compactive efforts in the laboratory. This was accomplished by preparing several bituminous mixtures, both sand asphalts and asphaltic concretes, at the approximate design optimum asphalt contents. Specimens of each mixture were compacted by (a) the original procedure, (b) by 10 blows of the 12.5-lb. hammer with 3-7/8 in. foot, and (c) by 10, 15, and 20 blows of the 10-lb. hammer with the 3-7/8 in. foot. The data for these tests are not presented here: however, from an analysis of the results it was concluded that 15 blows of the 10-lb., 3-7/8 in. hammer could be used for construction control of bituminous pavements.

As a final measure, it was considered that the 15-blow compactive effort (10-lb. 3-7/8 in. hammer) might be confused with the 50-blow compaction procedure (10-lb., 3-7/8 in. hammer) used in design. Also,

TABLE 6

Comparison of Field and Laboratory Densities
Stockton Test Section No. 2, California

Design Number	Course	Unit Weight -- Lb. Per Cu. Ft. Field	Laboratory
1	Binder	156	149
2	Binder	156	157
3	Wearing	151	151
4	Wearing	149	153
5	Binder	151	154
6	Wearing	146	152

TABLE 7

Correlation of Laboratory Densities,
15-Blow and 50-Blow Compaction

	Density							
	Per Cent Maximum Theoretical				Unit Weight Total Mix			
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Identification	15-Blow Per Cent	50-Blow Per Cent	Diff. Per Cent	(1) / (2) Per Cent	15-Blow Lb	50-Blow Lb	Diff. Lb	(5) / (6) Per Cent
				Sand Asphalt				
Blend A	92.0	94.7	2.7	97.1	136.9	141.0	4.1	97.1
Blend 3	90.3	93.4	3.1	96.7	133.9	138.5	4.6	96.7
Blend 4	94.5	96.1	1.6	98.3	142.3	144.8	2.5	98.3
Blend 5	93.1	95.2	2.1	97.8	141.7	144.8	3.1	97.8
Blend 8	91.6	93.9	2.3	97.6	137.7	141.1	3.4	97.6
				Asphaltic Concrete				
Blend B (Gravel)	94.3	95.6	1.3	98.6	144.7	146.7	2.0	98.6
Blend B (Slag)	94.9	96.8	1.9	98.0	141.7	144.5	2.8	98.0
Blend B (Crushed Limestone)	93.6	95.5	1.9	98.0	148.2	151.3	3.1	98.0
Mix 11 (Crushed Limestone)	94.4	95.7	1.3	98.6	147.6	149.7	2.1	98.6
Blend 10 (Gravel)	92.8	94.6	1.8	98.1	142.5	145.2	2.7	98.1
Blend 12 (Gravel)	93.9	95.3	1.4	98.5	143.8	145.9	2.1	98.5
Approximate Average			2.0	98.0			3.0	98.0

the use of two compactive efforts requires additional laboratory work. Therefore, analyses were made which are intended to provide a method for the control of construction densities based entirely on the compaction effort used for design. Five sand asphalt and six asphaltic concrete mixtures were prepared in the laboratory at their design optimum asphalt contents. Specimens of each mixture were compacted using both 15 and 50 blows of the 10-lb. hammer, 3-7/8 in. diameter foot. The densities of compacted specimens from these tests are shown on Table 7, both as percent of maximum theoretical density and as unit weight in lb. per cu. ft. The difference in densities for the two compactive efforts on each mixture was obtained and expressed as a percentage of the 50-blow density used for design. Although the indicated percentage values vary somewhat, the variation is small. On the average, the density of a specimen compacted to 15 blows will be 98 percent of the density obtained with 50-blow compaction. It is considered therefore, that good construction rolling shall be stipulated as that amount which produces a density in the pavement equivalent to 98 percent of that determined by the 50-blow compactive effort used for design.

CONCLUSIONS

Based on the results of analyses and investigations presented in this paper, the following conclusions are made:

a. The laboratory compactive effort for use in design of asphalt paving mixtures should consist of 50 blows on each side of the specimen, using a 10-lb. hammer with 18 in. drop and a 3-7/8-in. diameter foot.

b. Good construction rolling can be considered that effort which produces a density in the asphalt pavement equal to 98 percent of that secured by the 50-blow laboratory compaction.

DETAILED TEST PROCEDURES FOR DESIGN AND FIELD CONTROL OF ASPHALT PAVING MIXTURES

By

JOHN M. GRIFFITH*

This paper delineates the test procedures which have resulted from the comprehensive investigation outlined in the preceding papers of this symposium. The apparatus used in the laboratory is described and the method of selecting design asphalt contents from the test data is discussed. Since this test method is also adaptable for use in the construction control of plant mixtures and for determining the properties of in-place pavements, by means of core samples, a description of these features is included. A list of the equipment required to conduct the tests subsequently outlined is shown on Table 1. Detailed working drawings of the compaction and testing equipment are shown on Figures 1-5 inclusive.

The completed assembly of Marshall stability testing equipment mounted in the CBR frame is shown on Figure 5. This testing equipment consists of the CBR testing frame in which is mounted a proving ring with gage dial, screw-jacking mechanism, the flow meter and Marshall stability breaking head, and a penetration piston used as an extension by which to transfer load applied by jack to the proving ring.

LABORATORY TEST FOR DESIGN OF ASPHALT PAVING MIXTURES

Penetration of asphaltic mixture - When using the method to design a bituminous pavement mix, it is first desirable to make a sieve analysis and to determine the specific gravity of the aggregates and filler proposed for use. Specific gravity of the asphalt cement likewise should be determined for use in computations as discussed later. The proper proportions of various types of aggregates and filler to produce a reasonable gradation may then be determined. Depending on the quantity of the material to be produced and local costs of various aggregates, it may be desirable to investigate a number of aggregate blends.

To insure accurate control of blends in the preparation of test mixtures, the aggregate should be separated into fractions, and where adequate heating facilities are available the following size separations are suggested: 3/4 in., ½ in., 3/8 in., Nos. 4, 10, 40, 80, 200. Aggregate larger than 1 in. should not be used in the standard equipment with this test method.

All separated fractions of aggregate and filler should be heated separately to temperatures between 350 F. and 375 F. Asphalt cement should be heated to temperatures between 250 F. and 280 F., but should not be held at this temperature for more than one hour. Figure 6 shows aggregate heating facilities at the Waterways Experiment Station. After all materials have reached the desired temperature, the bowl or pan in which the mixture is to be prepared is placed on a solution balance and tared. The aggregate and filler are then scooped from the heating pans and weighed in proportions calculated to give approximately 3000 gms of the desired blend. When removing the aggregates from the heating pans, a representative sample of the material may be obtained by scooping to the bottom of the pan; otherwise segregation may occur. The aggregate and filler are then thoroughly mixed by a trowel or large spatula. Aggregate temperatures should then be be-

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TABLE 1

EQUIPMENT REQUIREMENTS FOR LABORATORY TESTS

Item	No Req'd	Remarks
Compaction mold cylinder	6	See Figure 1
Compaction mold base plate	4	See Figure 1
Compaction mold collar	2	See Figure 1
Compaction hammer	2	See Figure 2
Marshall breaking head	1	See Figure 3
Flow meter	1	See Figure 3
Testing machine	1	See Figures 4 and 5
Sample extractor	2	See Figure 1
Ovens or hot plates for heating aggregate, asphalt cement and molding equipment		Approximately 12-sq-ft heating surface area (approximately 1 5-ft by 8-ft) is desirable
Pans for heating aggregate	12	Approximately 12-in x 18-in x 4-in
Can with pitcher-type handle for heating asphalt cement	2	1-gallon capacity
Mixing bowls or pans for mixing aggregate and asphalt cement	2	Approximately 10-quart capacity To fit mechanical mixer if available
Mechanical bakery or restaurant-type mixer	1	Approximately 10-quart capacity
Scoop for handling hot aggregate	1	2-quart size
Square pointed masons trowels	2	2-in x 4-in blade, wood handle
Spatulas	2	1-in x 6-in blade, wood handle
Thermometers for determining mixture temperatures	6	Dial-type with metal stem or armored glass thermometer, minimum sensitivity 5-degree F , range 50-degree F to 400-degree F
Compaction pedistal, support for compaction mold while tamping	1	6-in x 6-in wood post capped with 12-in x 12-in x 2-in wood, and 12-in x 12-in x 1-in steel plate, supported on concrete base or floor slab
Hot water bath with perforated false bottom for heating test specimens, thermostatically controlled for 140-degree F ± 1-degree	1	Approximately 18-in x 30-in x 9-in deep
Thermometers for hot water bath	2	Mercury thermometers, 0 2-degree F divisions, 134-degree F to 148-degree F range
Solution balance for weighing aggregate and asphalt	1	20-kg capacity, sensitive to 1 gm
Balance for weighing compacted specimens	1	2-kg capacity, sensitive to 0 5-gm
Saddle and wire basket for weighing specimens under water	1	
Water bucket for weighing specimens under water	1	Approximately 10-quart capacity
Welders gloves, or similar, for handling hot equipment	3 pr	

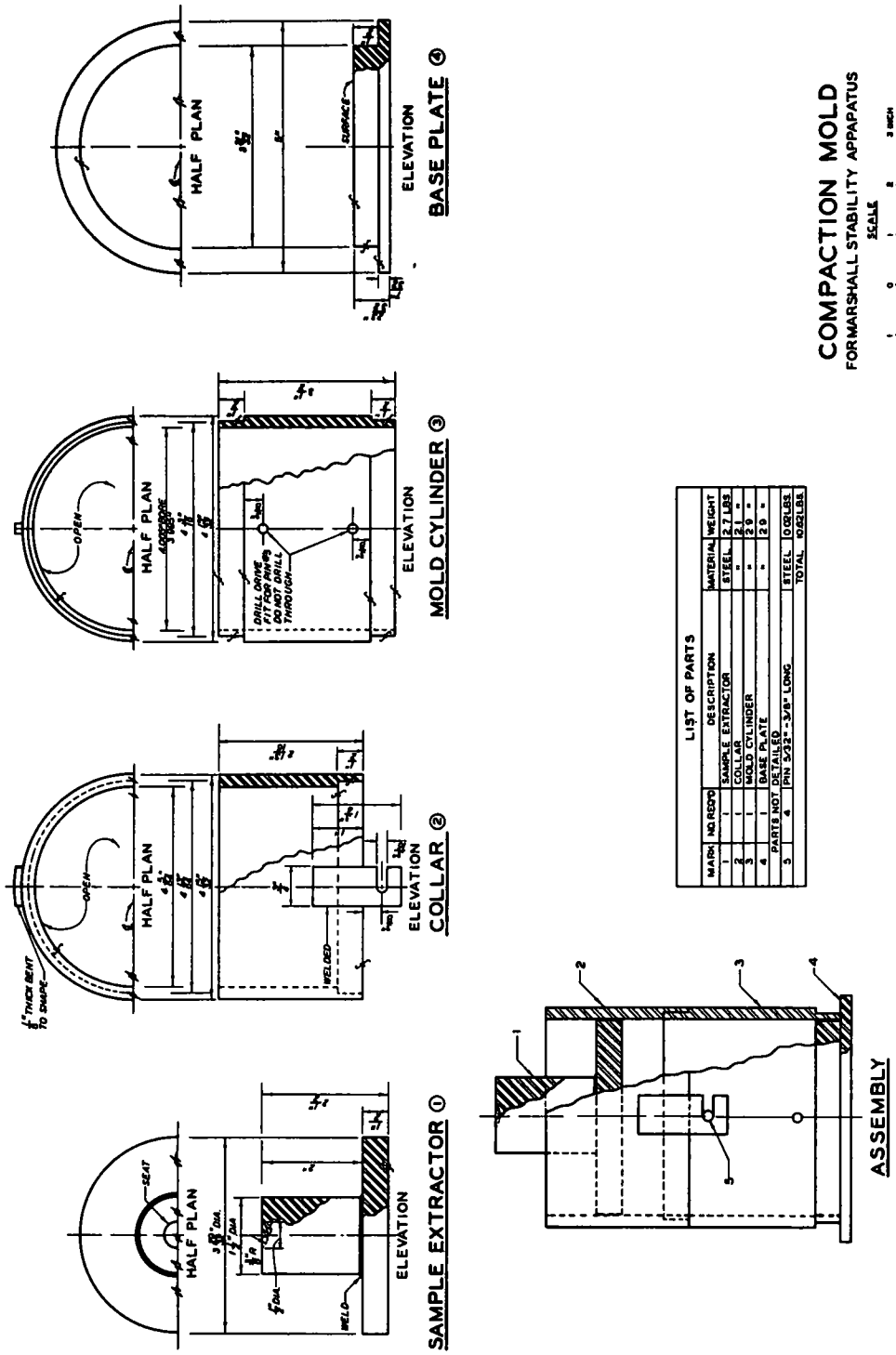
tween 340 F. and 360 F. Upon attaining the desired mixing temperature, a crater is formed in the mixing bowl or pan. The bowl (or pan) and aggregates are rebalanced on the solution balance and the hot asphalt cement is introduced in the required amount.

The amounts of asphalt cement used in the preparation of test specimens necessarily must be estimated, since one of the primary objectives of the test method is to determine the optimum asphalt content. An estimate of the optimum asphalt content based on judgement and past experience with similar mixtures is adequate for a starting point. Trial mixtures are prepared at the estimated optimum asphalt content and generally at asphalt contents 1 and 2 percent below and above the estimated optimum asphalt content. Experience has indicated that eight test specimens are required at each asphalt content to assure adequately accurate test data.

Mixing is accomplished immediately after the introduction of the asphalt cement and should be completed as rapidly as possible. Mixing may be done either by hand or in a mechanical mixer. A 10- to 14-quart bread dough mixer is recommended. Thorough mixing should be accomplished within two minutes. The temperature of the mixture should not be below 225 F. upon completion of mixing. If below this temperature, the mixture should be discarded and the process repeated. The mixture should not be reheated after mixing.

Preparation of test specimens - Production of test specimens is initiated immediately after mixing is completed. The compaction hammers and compaction molds should be heated to between 200 and 300 F., cleaned and ready for use. All of the mixture is first transferred from the mixing bowl to a large pan, divided equally, and each half placed in a compaction mold. A piece of filter paper or paper toweling, cut to size and placed in the bottom of the mold before the mixture is introduced, facilitates removal of the base plate after compaction. After the mixture is transferred to the molds, compaction proceeds immediately.

The temperature of the mixture immediately prior to compaction should not be less than 225 F. After the mix has been



LIST OF PARTS

MARK	NO REQ'D	DESCRIPTION	MATERIAL	WEIGHT
1	1	SAMPLE EXTRACTOR	STEEL	2.7 LBS
2	1	COLLAR	"	2.1 "
3	1	MOLD CYLINDER	"	2.9 "
4	1	BASE PLATE	"	2.9 "
PARTS NOT DETAILED				
5	4	PIN 5/32" - 3/8" LONG	STEEL	0.01LBS
			TOTAL	10.62LBS

Figure 1. Plan and Profiles of Test Section

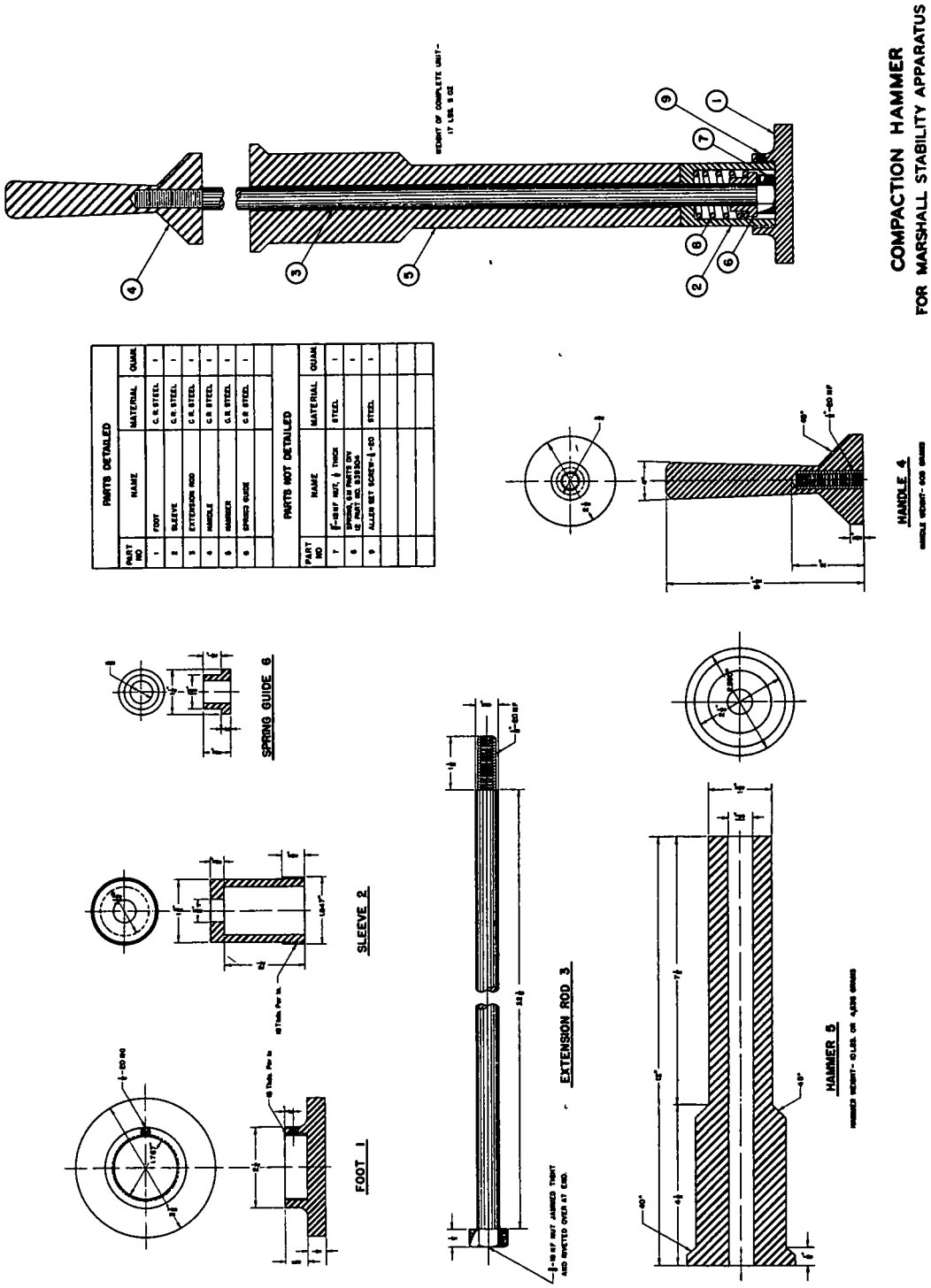

COMPACTION HAMMER
FOR MARSHALL STABILITY APPARATUS
PARTS DETAILED
PART NO | NAME | MATERIAL | QUAN.
1 | FOOT | C.R. STEEL | 1
2 | SLEEVE | C.R. STEEL | 1
3 | EXTENSION ROD | C.R. STEEL | 1
4 | HANDLE | C.R. STEEL | 1
5 | HAMMER | C.R. STEEL | 1
6 | SPRING GUIDE | C.R. STEEL | 1
PARTS NOT DETAILED
PART NO | NAME | MATERIAL | QUAN.
7 | 1/2-13 NF HEX. NUT, 1/2 THICK | STEEL | 1
8 | SPRING, SEE PARTS DIV. IS PART NO. 5.33.504 | | 1
9 | ALLEN SET SCREW-1/4-20 | STEEL | 1
HEIGHT OF COMPLETE UNIT - 17 LBS. 5 OZ
FOOT 1
SLEEVE 2
SPRING GUIDE 6
EXTENSION ROD 3
HAMMER 5
HANDLE 4

Figure 2.

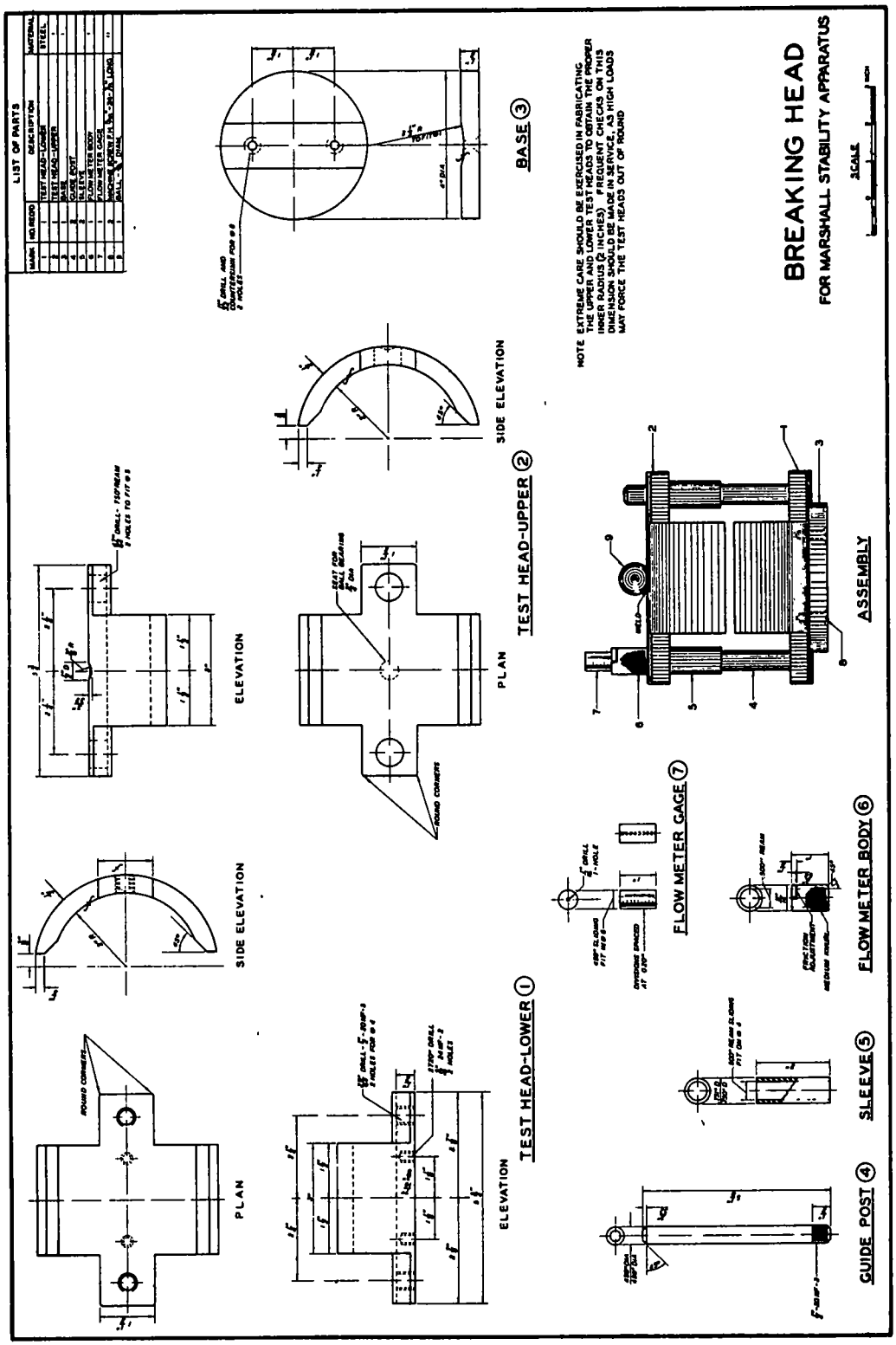
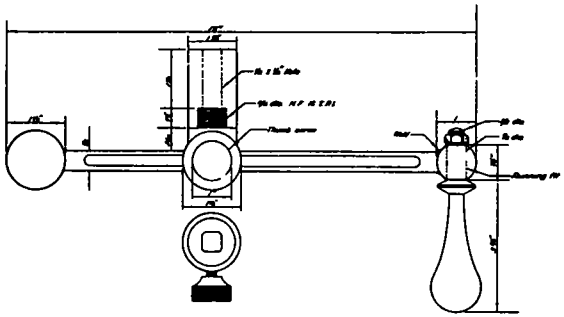
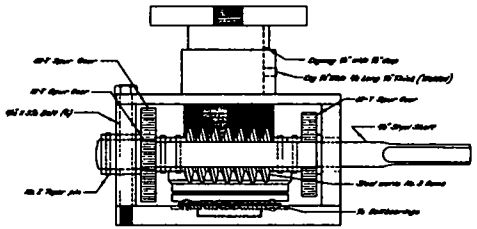

LIST OF PARTS
MARK
NO. REQ'D
DESCRIPTION
MATERIAL
TEST HEAD-LOWER
TEST HEAD-UPPER
BASE
GUIDE POST
TEST HEAD
FLOW METER BODY
FLOW METER GAGE
SHOULDER SCREW
BALL
STEEL
BREAKING HEAD
FOR MARSHALL STABILITY APPARATUS
SCALE
NOTE EXTREME CARE SHOULD BE EXERCISED IN FABRICATING THE UPPER AND LOWER TEST HEADS TO OBTAIN THE PROPER INNER RADIUS (2 INCHES) FREQUENT CHECKS ON THIS DIMENSION SHOULD BE MADE IN SERVICE, AS HIGH LOADS MAY FORCE THE TEST HEADS OUT OF ROUND
BASE ③
SIDE ELEVATION
TEST HEAD-UPPER ②
PLAN
ELEVATION
SIDE ELEVATION
ASSEMBLY
TEST HEAD-LOWER ①
ELEVATION
PLAN
FLOW METER GAGE ⑦
FLOW METER BODY ⑥
SLEEVE ⑤
GUIDE POST ④

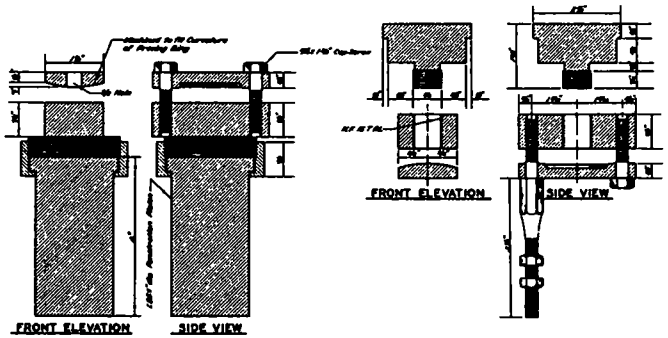
Figure 3



ADJUSTABLE JACK CRANK

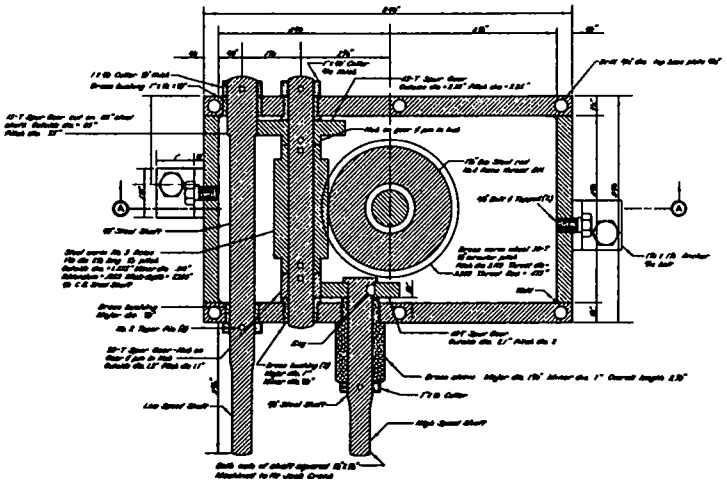


SIDE VIEW
(END PLATE REMOVED)

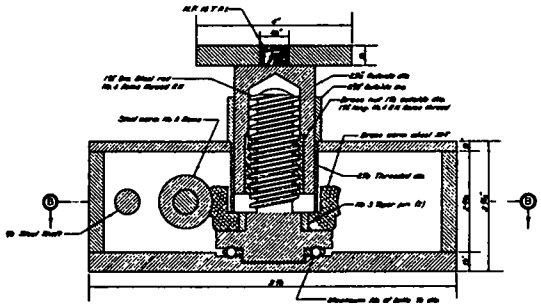


FRONT ELEVATION
SIDE VIEW
(LOWER)
FRONT ELEVATION
SIDE VIEW
(UPPER)

DETAILS OF UPPER AND
LOWER PROVING RING ATTACHMENTS



SECTION B-B



SECTION A-A

DETAILS OF SCREW JACK AND
PROVING RING ATTACHMENT
FOR MARSHALL STABILITY APPARATUS

Figure 4

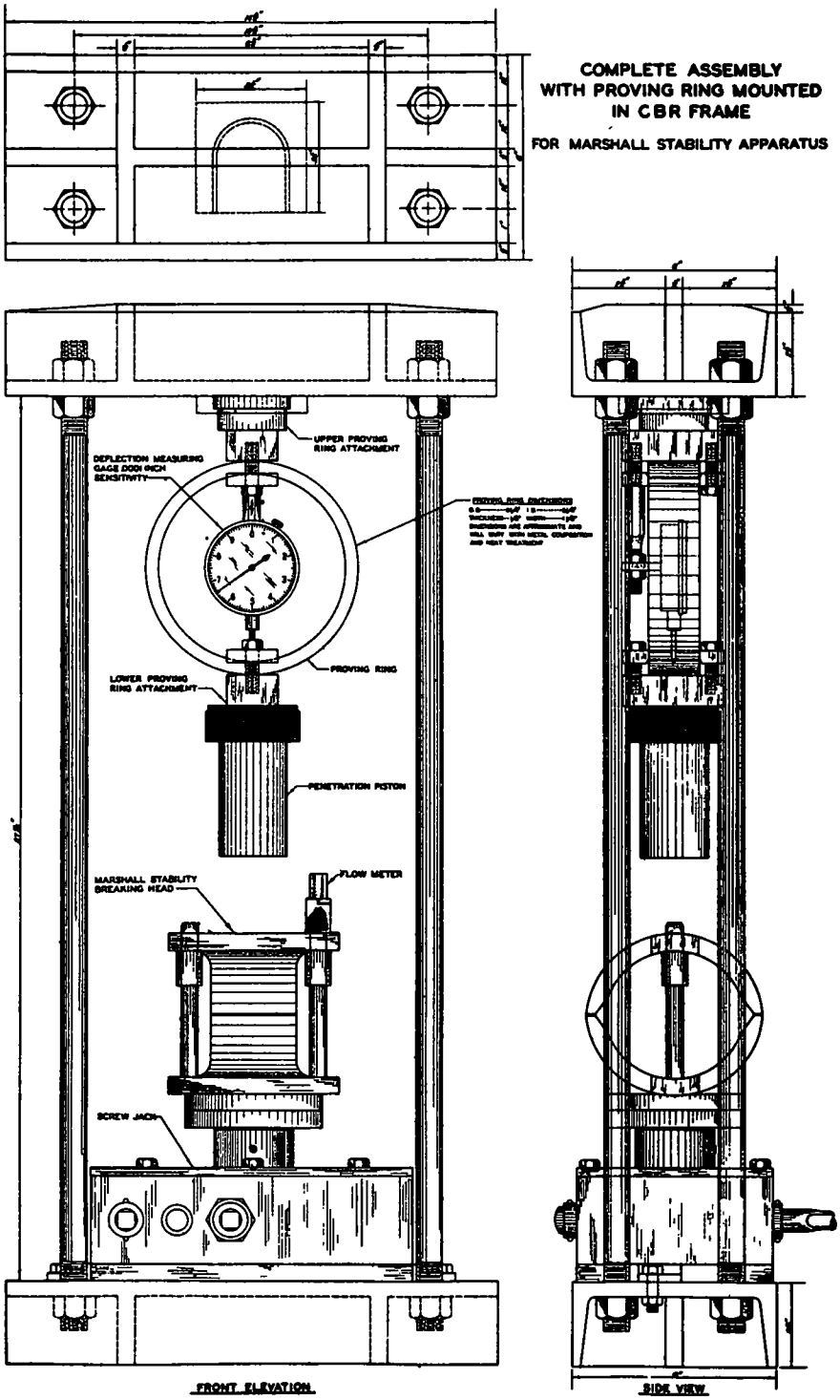

COMPLETE ASSEMBLY
WITH PROVING RING MOUNTED
IN CBR FRAME
FOR MARSHALL STABILITY APPARATUS
UPPER PROVING RING ATTACHMENT
DEFLECTION MEASURING GAGE 0001 INCH SENSITIVITY
PROVING RING
LOWER PROVING RING ATTACHMENT
PENETRATION PISTON
FLOW METER
MARSHALL STABILITY BREAKING HEAD
SCREW JACK
FRONT ELEVATION
SIDE VIEW

Figure 5

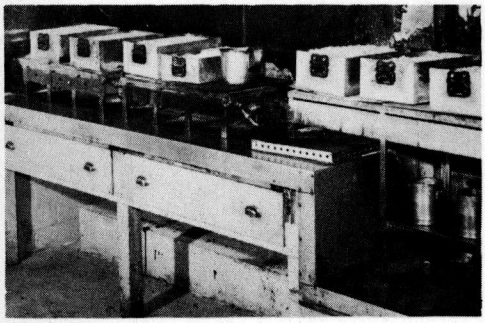


Figure 6. Aggregate and Asphalt Heating Facilities

placed in the mold, the collar is removed and the surface of the material smoothed with a trowel to a slightly rounded shape. The collar is then replaced and the surface of the mix leveled using hand pressure on a heated sample extractor (Figure 7). An extra sample extractor equipped with a wooden handle extension is handy for this purpose. The mold assembly is placed on a heavy substantial compaction base, the heated compaction hammer is placed on the specimen and 50 blows of the hammer are applied. After this the base plate and collar are removed and the mold reversed and reassembled so that the base plate is adjacent to the original top of the specimen. Fifty blows of the compaction hammer are then applied to this face of the specimen.

The base plate and collar are removed and the mold with the specimen inside is immersed in cool water for approximately two minutes, after which the collar is replaced on the mold and the sample extractor is placed on the opposite end of the specimen. The assembly is then placed with the mold collar down in the compression machine, and pressure is applied to the sample extractor, forcing the specimen into the mold collar. The specimen may then be removed from the mold and suitably identified. It should be carefully handled and placed on a smooth and level surface until ready for testing. The height of the specimen should be 2½ in. ± 1/8 in.

Testing specimens - Specimens may be tested at any time after preparation. Weights are determined for each specimen by weighing in air and suspended under water (to obtain the volume). The water should not contain an excess of suspended or dissolved materials and its temperature should be approximately 77 F. The volume of specimens having an open texture is determined by measuring their height and diameter as accurately as possible or by coating with paraffin.

The specimen is immersed in a water bath at 140 F. ± 1 deg. for a period of at least 20 min. After this period it is ready to be tested for stability and flow in the Marshall apparatus; however, testing should not be begun until all apparatus is in readiness, as follows:

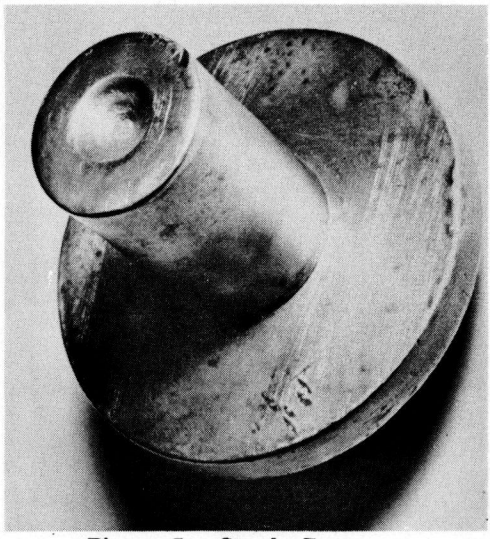


Figure 7. Sample Extractor

a. The inside surfaces of the upper and lower test heads and the guide rods of the breaking head should be thoroughly cleaned, the guide rods well lubricated, and the upper test head should slide freely over the guide rods to the lower test head.
b. Clearance between the jack and the lower proving ring support should be just sufficient to permit introduction of the test mold.

After the necessary preparations have been completed, the specimen is removed from the hot water bath and fitted to testing position on its side in the lower part of the breaking head; the complete assembly is then placed in testing position in the compression machine. The flow meter is placed on one of the guide rods and pressed down against the upper test head, and the initial reading of the flow meter is made and recorded. Pressure is then applied to the specimen in such a manner that the jack head rises at a rate of 2 in. per min. Failure of the specimen occurs and is recorded when the load-measuring dial reaches its maximum reading and begins to return toward zero. The total number of pounds required to produce failure of the specimen is recorded as its stability value. In order to prevent excessive cooling of the specimen with a resulting increase in stability value, the entire test procedure from the time the specimen is removed from the water bath should be performed as quickly as possible; normally, the test should be performed in about 30 sec. Figure 8 shows details of a specimen ready for test in the field apparatus. A close-up view of the specimen, test head and flow meter is shown on Figure 3 of a preceding paper entitled "Selection of Test Equipment."

The flow value is obtained during the test for stability. When the load is being applied to the specimen, the body of the flow meter should be held firmly against the top of the upper test head so that the guide rod pushes the flow meter gage upwards as the sample deforms. When the maximum stability reading is obtained on the load measuring dial, the flow meter is instantly removed from its position on the guide rod. The difference between the initial and final readings expressed in hundredths of an inch is recorded as the flow value.

The stability test may be performed in a universal testing machine with stress-strain recorder as shown on Figure 9. The stability is recorded as the maximum value on the load-deformation curve. The flow value read from the curve is selected at the point beyond the peak where the load first begins to decrease. It has been established that this flow value agrees closely with the flow meter reading, as there is a slight time lag in removing the flow meter from the breaking head in the hand method of performing the test.

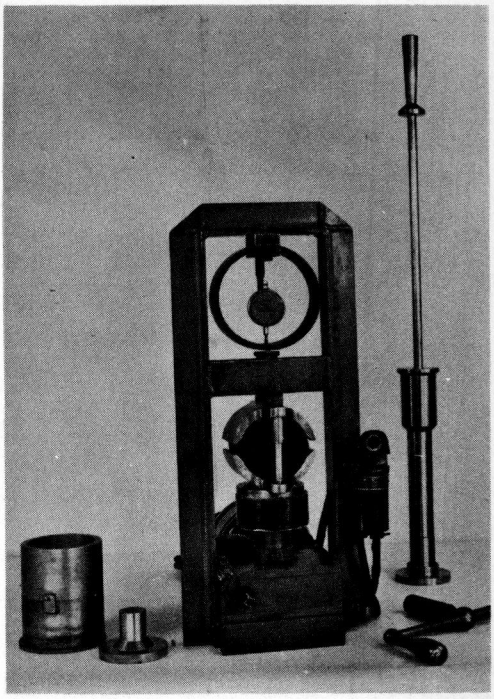


Figure 8. Marshall Stability Apparatus for Field or Laboratory Use

INTERPRETATION OF TEST RESULTS

Test properties curves - Data obtained as outlined above furnish the basis for determining, either directly or by computation, the following properties of each test specimen:

a. Flow
b. Stability
c. Unit weight, total mix
d. Unit weight, aggregate only
e. Percent voids, aggregate only
f. Percent voids, total mix
g. Percent voids filled with asphalt cement

Data averages from the eight specimens at each asphalt content are then prepared

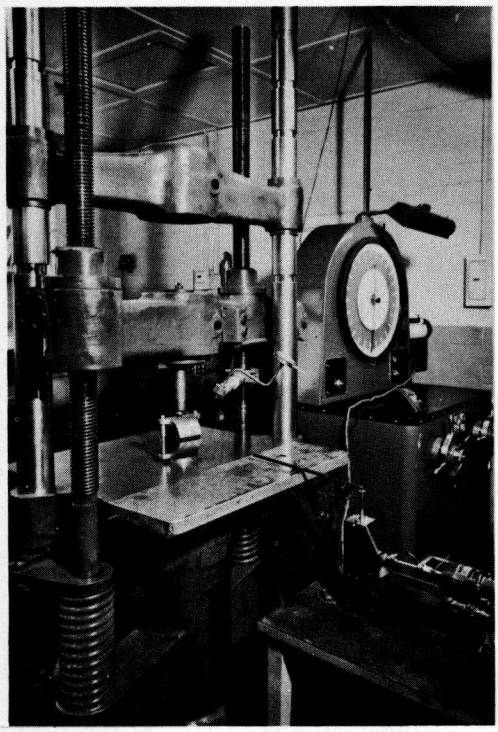


Figure 9. Universal Testing Machine Used for Marshall Test

for each of the above test properties. Individual test values which are at considerable variance with the average may be discarded. The average test properties outlined above are then plotted versus asphalt content on separate diagrams and smooth curves are drawn through the plotted points. For the determination of optimum asphalt, the unit weight, aggregate only, and percent voids, aggregate only, are not used as criteria. These data are frequently computed for general information only. The other five test properties are used in the determination of optimum asphalt, and typical data plots of these five properties are shown on Figure 10. In order to eliminate erratic test values, it has been found convenient to plot the test points for the unit weight of the total mix and to draw the best smooth curve through these points. The remaining density and void relationships are then computed from values read from this curve. In this manner, smooth curves are obtained for the computed test properties and all void and weight relationships are in mutual agreement.

Selection of optimum asphalt content - The test procedure and computations described previously have been directed toward furnishing information on a given bituminous mixture such that the proper asphalt content may be selected for satisfactory pavement design. The asphalt content desired, termed the "optimum asphalt," is determined by assigning criteria to certain of the test properties, selecting the asphalt content that satisfies each individual case, and averaging the asphalt contents obtained. The average value is the optimum asphalt content. The criteria for satisfactory pavements have been established by the investigations described in the preceding papers and are found on page 87.

The criteria shown above for asphaltic concrete are considered to be entirely valid while criteria shown for sand asphalt are considered to be tentative and subject to possible revision. This is due to the quantity and type of data obtained to date on these two types of asphaltic mixtures. The test section, described in a previous paper, and other field data, have given an adequate background for the selection of asphaltic concrete design criteria while data on the sand asphalt pavements have been limited.

An example of the selection of optimum asphalt content is shown for the test results plotted on Figure 10. Using the criteria for asphaltic concrete presented in the preceding paragraph the following asphalt contents have been selected for the various test properties:

Test Property	Selected Asphalt Content-Percent
Stability	5.3
Unit Weight Total Mix	5.5
Voids Total Mix	5.8
Voids Filled With Asphalt	6.3
Average	5.7

Design Criteria

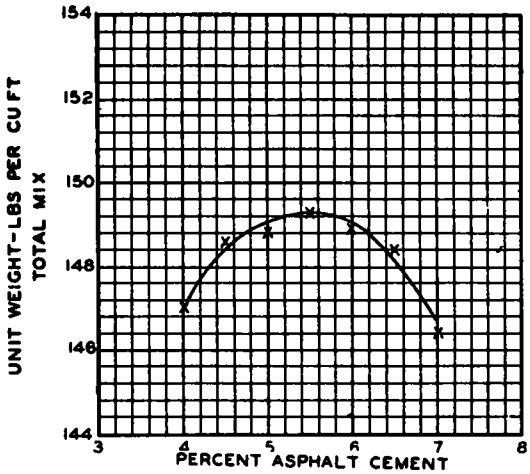
Test Property	Limits	Value To Be Used For Selection of Optimum Asphalt Content
Asphaltic Concrete		
Flow	Less than 20	-
Stability	More than 500	Maximum
Unit Weight, Total Mix	-	Maximum
Percent Voids, Total Mix	3 to 5	4
Percent Voids Filled with Asphalt	75 to 85	80
Sand Asphalt		
Flow	Less than 20	-
Stability	More than 500	Maximum
Unit Weight, Total Mix	-	Maximum
Percent Voids, Total Mix	5 to 7	6
Percent Voids Filled with Asphalt	65 to 75	70

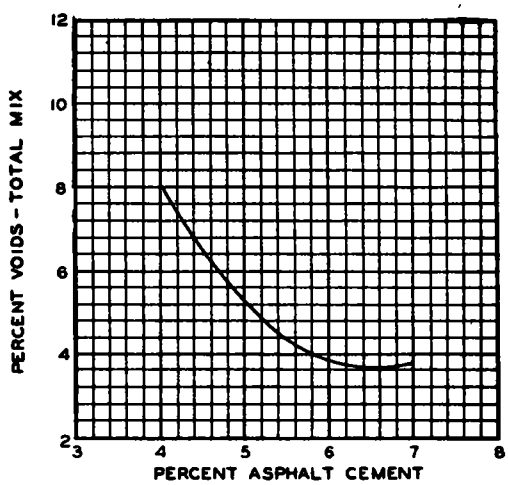
The individual test properties at the average asphalt content of 5.7 percent are then reexamined to determine how closely they agree with the criteria. At this asphalt content the flow value is 14, stability is 700 lb., voids total mix is 4.1 percent, and voids filled with asphalt 76 percent. All values are in reasonable agreement with the criteria. The variables that are present during construction are recognized. The value of 5.7 percent does not imply absolute accuracy but may vary within a range of values for the criteria. For example, inspection of the test properties curves on Figure 10 shows that at asphalt contents between 5.5 and 6 percent the individual values are in substantial agreement with the criteria, and any value between these limits may be acceptable for construction. However, the asphalt content selected on the basis of test properties should be used for design purposes.

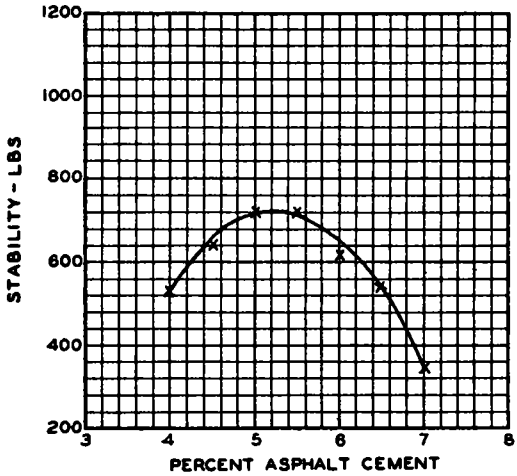
In some cases the selection of an optimum asphalt content from the test properties curves is more difficult than was shown in the example cited. Certain mixes, for instance, may approach but not reach 4 percent voids total mix or 80 percent voids filled with asphalt. If the gradation of the mix and the other test properties are otherwise acceptable, a tolerance of 1 percent in the voids total mix and 5 percent in the voids filled with asphalt may be allowed. The optimum asphalt content as determined from stability and unit weight criteria is examined with respect to the voids total mix and voids filled with asphalt; if these values at optimum asphalt are within the tolerances allowed the mix is considered satisfactory. If the values are not within the tolerances, consideration should be given to adjusting the optimum asphalt to come within the voids tolerances, provided this asphalt content is reasonable with respect to maximum stability and unit weight and the flow does not exceed 20. If the selected optimum asphalt content does not provide test properties that are in reasonable agreement with the criteria, a redesign of the blend is indicated.

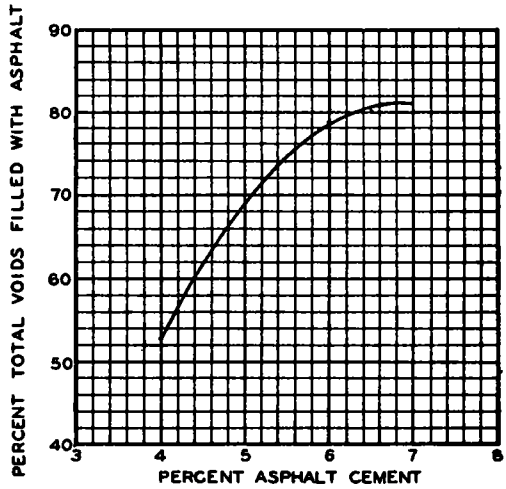
LABORATORY TESTS FOR FIELD CONTROL OF ASPHALT PAVING MIXTURES

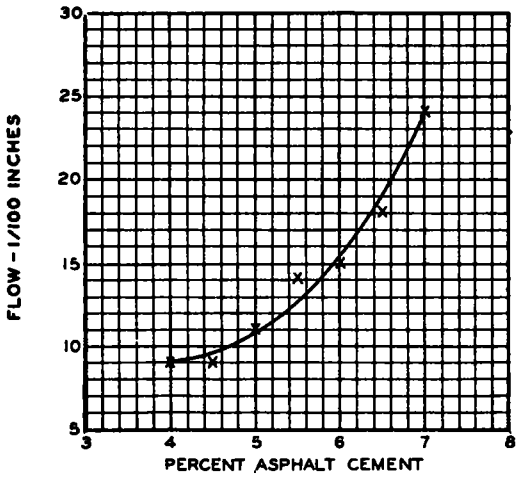
The foregoing paragraphs have described the method by which the proper asphalt content for the design of a bituminous pavement is obtained in the laboratory. Fully as important as the initial design procedure is the control of plant operations and the placement of the mixture in the field to insure that the pavement; as constructed, satisfies the design requirements. The following paragraphs outline a suggested procedure for the control of bituminous mixtures at the plant and in the field. It is recommended that ade-











TYPICAL CURVES
TEST PROPERTIES VS
ASPHALT CONTENT
ASPHALTIC CONCRETE

Figure 10

quate laboratory facilities be provided at the plant in order that proper control may be exercised. The test procedure is not intended to supplant the routine gradation and extraction tests that are normally run in connection with plant control.

Control of plant mixtures - It is probable that initial laboratory tests to determine the proper proportions of materials and the optimum asphalt content will be conducted on aggregate obtained from stockpiles or from proposed source locations. Such materials, when they are processed through the plant, often are subject to changes in gradation primarily due to degradation of aggregate, difficulties in securing representative aggregate samples, nonuniformities in the supply of material or loss of fines in the driers. Such changes may require a different proportioning of the aggregate in the plant to meet specification requirements, and possibly a modification of the asphalt content may be required to meet the pavement design criteria. The first step in plant control, therefore, is to obtain representative samples of the processed aggregate (preferably samples from the bins after the aggregate has been processed through the plant) and to adjust the proportions of material as may be desired. The test procedure for selection of optimum asphalt content should then be repeated to determine what changes, if any, are necessary in the optimum asphalt content. The separation of aggregate into numerous fractions for laboratory blending is too tedious and possibly unnecessarily accurate for plant control. For this purpose it is satisfactory to separate the coarse and fine aggregates on the No. 10 sieve and further separate the coarse aggregate on the ½-in. sieve. The normal screen sizes used on the plant may also prove satisfactory for separation of aggregates. This method will give more variations in blend proportions due to variations in gradation of the material, but it is believed sufficiently accurate for plant control purposes.

After the plant is in operation, frequent checks should be made to insure that the bituminous mixture as produced meets the design requirements. Representative specimens of the plant-mixed material sufficiently large to make 8 test specimens (about 50 lb.) should be obtained and specimens compacted as previously outlined. Samples should be compacted before the mixture has cooled below 225 F., and the mix should not be reheated. The completed specimens are then tested and test properties are determined. The number of representative samples prepared and tested in this manner will vary with the size of the job. It is suggested that continuous tests be made in the first few days of operation in order to determine the variation in test results due to the normal variations in the stockpiled aggregate and in plant operations. Routine checks may be made at less frequent intervals when plant operations are stabilized.

A comparison of test results on the plant mixed materials with those obtained in the design tests will indicate whether any significant changes in aggregate gradation or asphalt content have taken place which will affect the pavement design. An increase in the flow value to above 20, a decrease in stability of 50 to 100 lb., variations in voids total mix greater than 1 percent and voids filled with asphalt greater than 5 percent indicate the need for revisions in the proportions of aggregate, the asphalt content, or both. The values cited are tolerances which are considered reasonable; specific tolerances may be established for a given job. Since speed is essential in the proper control of plant mixtures, considerable time may be saved by computing only the flow, stability, and unit weight total mix of the test specimens. For a given mixture, variations in the unit weight total mix reflect variations in the voids relationships. For instance, a variation of 1.5 to 2 lb. per cu. ft. in the unit weight total mix is accompanied by about the 1 percent change in voids total mix and the 5 percent change in voids filled with asphalt mentioned above. The allowable tolerance in unit weight for a given voids tolerance may be computed and used for rapid control

of the plant mixture.

Control of field construction - Field control of placement of bituminous pavements is based on attaining a desired density in rolling operations. Results of previous studies, discussed in a preceding paper, indicate that a laboratory compactive effort of 15 blows on each side of the test specimen will produce densities approximately equivalent to those obtained with carefully controlled rolling in the field. It is further shown that the density obtained by 15-blow compaction approximately equals 98 percent of the density obtained with 50-blow compaction. Therefore, a factor amounting to 98 percent of the density determined from test specimens compacted by the 50-blow procedure previously described should be computed.

OBTAINING AND TESTING PAVEMENT SAMPLES

Pavement density control - To control the desired density to be obtained in the field, test specimens are cored or otherwise cut from the pavements during construction. The desired construction density being known, density measurements on these specimens indicate whether additional rolling is required. Specified construction densities may be easily obtained or even exceeded in some mixtures, whereas in others careful control in the rolling procedures must be exercised.

Field coring of pavements - Coring field test specimens may be accomplished with truck-mounted rotary core drilling equipment, such as that shown on Figure 11, provided with a means of supplying water to the area being cored to flush out the cuttings. A steel core barrel tipped with carboloy chips (Figure 12) has been found to be very satisfactory for most coring operations in asphaltic pavements. During hot weather it has been found necessary to chill the pavement with ice prior to coring. Field cores should be 4 in. ± 1/16 in. diameter. Where density measurements only are desired or where core cutting facilities are available in the laboratory but not in the field, square or rectangular segments of pavement may be cut by a mattock, or other means.

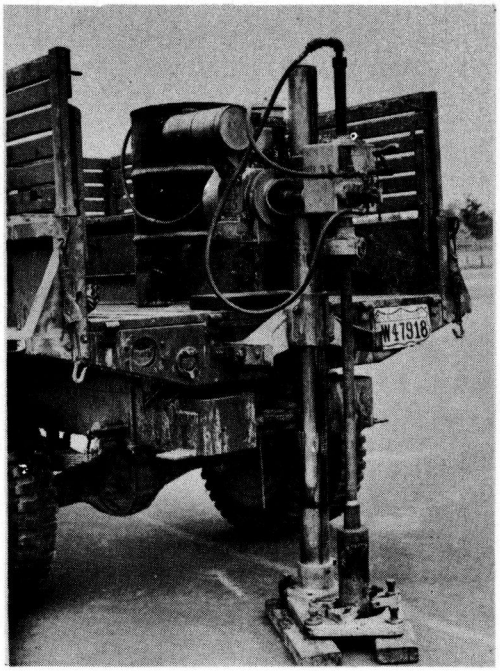


Figure 11. Core Drill Rig

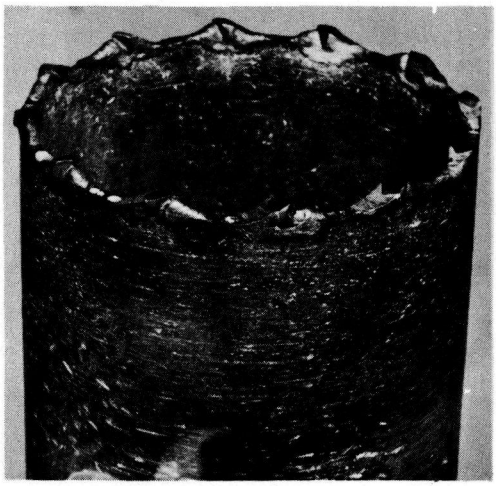


Figure 12. Detail of Asphalt Drill Bit

Preparation of cores for testing - Field cores should be split at the interface between construction layers prior to testing. To accomplish this splitting a heated knife is drawn around the circumfer-

TABLE 2

STABILITY CORRELATION RATIO

Volume of Specimen in Cubic Centimeters	Approximate Thickness of Specimen in Inches	Correlation Ratio
200 - 213	1	5.56
214 - 225	1-1/16	5.00
226 - 237	1-1/8	4.55
238 - 250	1-3/16	4.17
251 - 264	1-1/4	3.85
265 - 276	1-5/16	3.57
277 - 289	1-3/8	3.33
290 - 301	1-7/16	3.03
302 - 316	1-1/2	2.78
317 - 328	1-9/16	2.50
329 - 340	1-5/8	2.27
341 - 353	1-11/16	2.08
354 - 367	1-3/4	1.92
368 - 379	1-13/16	1.79
380 - 392	1-7/8	1.67
393 - 405	1-15/16	1.56
406 - 420	2	1.47
421 - 431	2-1/16	1.39
432 - 443	2-1/8	1.32
444 - 456	2-3/16	1.25
457 - 470	2-1/4	1.19
471 - 482	2-5/16	1.14
483 - 495	2-3/8	1.09
496 - 508	2-7/16	1.04
509 - 522	2-1/2	1.00
523 - 535	2-9/16	0.96
536 - 546	2-5/8	0.93
547 - 559	2-11/16	0.89
560 - 573	2-3/4	0.86
574 - 585	2-13/16	0.83
586 - 598	2-7/8	0.81
599 - 610	2-15/16	0.78
611 - 625	3	0.76

NOTES: 1. The measured stability of a specimen multiplied by the ratio for the thickness of the specimen equals the corrected stability for a 2-½-inch specimen.

2. Volume-thickness relationship is based on a specimen diameter of 4 inches.

ance of the core at the interface of the layers making a groove approximately 1/16 in. deep. The core is then placed in an ice bath and allowed to chill thoroughly, after which it is removed from the bath and placed on its side on a level sur-

face. A heavy butcher knife or a machete is placed in the groove previously made and the back of the blade hit sharply with a hammer. It may be necessary to rotate the core and strike it at several points in the groove in order to break it apart.

Testing field cores - The cores are weighed, heated in the water bath, and tested in the Marshall apparatus as previously described. Since field cores are generally of some thickness other than the standard 2½-in. thickness to which laboratory compacted specimens are prepared, a correction must be applied to the stability value in order that all test results may be compared on a standard basis. Investigations made by the Waterways Experiment Station indicate there is a direct relationship between thickness and stability of specimens. Table 2 shows stability correction factors for specimens ranging in thickness from 1 to 3 in. Also presented are approximate volumes of 4-in. diameter specimens for the various thicknesses, as it is sometimes more convenient to use the volume determined by weighing the core in air and under water rather than to make an actual measurement of thickness. A correction factor for flow is not necessary.

COMPLETED CONSTRUCTION DATA

Upon completion of pavement structures, representative cored specimens of the pavement should be obtained and their test properties determined. Coring and testing periodically are also desirable to check the validity of the tentatively adopted criteria. It should be remembered that the criteria previously established were for airfield pavements supporting very heavy wheel loads. The frequency and nature of the traffic and the magnitude of the superimposed wheel loads on highways are different than for airfield pavements, and it may be that some modifications of these criteria for highway use are warranted. The maintenance of data and observation files on pavement behavior under highway traffic is the only sound basis on which to make such modifications. The pavement behavior study should include all locally potential sources of material to assure the establishment of satisfactory design criteria.

EVALUATION OF EXISTING PAVEMENTS

Coring and testing procedures previously outlined are also applicable to existing pavements. Much additional data of considerable value may be obtained for the verification of modification of proposed criteria by systematically evaluating the existing pavements. Locations in an existing road system may be chosen in which pavements are definitely failing, in which pavements are giving only slight indications of unsatisfactoriness, and in which pavements are definitely adequate. When such data are used to verify or modify the tentative criteria, the adequacy of the base and subgrade should definitely be established. Pavement criteria data should be used only where satisfactoriness or unsatisfactoriness is due to the properties of the pavement itself. Deficiencies of the base may, in some cases, be compensated for by the additional pavement thickness. This compensation, however, is discussed in another paper in this Symposium.

THE PRACTICAL APPLICATION OF THE DESIGN METHOD OF ASPHALTIC MIXTURES TO PAVEMENT CONSTRUCTION

by
W. K. BOYD*

The primary reason for the investigation which is the subject of this symposium was to provide methods for the solution of certain practical problems which are common to all asphalt paving projects. These problems may be listed as:

a. Are aggregate materials which are locally available suitable for use in asphalt pavements?

b. If two or more aggregates are to be blended, what are the desirable proportions for each?

c. What percent asphalt in the paving mixture should be used as a basis of estimate for total quantities required?

d. Is the asphalt mixture as produced and laid as a pavement of satisfactory quality, containing the proper amount of asphalt for the maximum intended service?

Items a, b, and c involve preliminary design work and should be completed well in advance of actual construction. Item d includes a final design based on materials actually taken from plant bins and includes constant sampling, testing, and analysis of the asphalt mixture as it is being produced.

With respect to items a, b, and c, the available aggregate may be restricted to a single source such as from a producer of commercial slag or a limestone rock quarry. In such cases the preliminary design may be limited to the preparation of a few mixes to determine the amount of filler and asphalt which might be required. In other cases a considerable choice of aggregate may be available, such as several local deposits of sand and gravel. In such cases it may be desirable to prepare a number of different trial mixes as a basis for selecting a final design which will provide for a satisfactory mix at the most economical price. For example, a local sand-gravel aggregate which is unsatisfactory by itself may be entirely satisfactory if blended with a very fine sand and a nominal percentage of a commercial filler. It is believed that the test procedures, the method of anaylsis, the test property criteria which have been outlined in previous papers provide adequate tools and a satisfactory method to solve the problems presented by items a, b, and c. Such preliminary studies take time and must be done well in advance of the construction period if the final plans and estimates are to reflect the results of the preliminary investigational work. The value of a reliable preliminary design, firm estimates as to quantities of material required, and prior approval of material for use is readily appreciated by responsible engineers familiar with the problem.

While the preliminary design and analytical work can probably be best accomplished in a central laboratory, the actual final pavement design used in the field and daily construction control tests (item d of first paragraph) must be done in a laboratory set up at the site of the mixing plant. With respect to a final mix design, it should be recognized that certain factors always are present which may cause modification of the preliminary design. Some of these factors include variation in gradation due to errors in sampling aggregate in the deposit or stock pile, changes made by crushing oversize material, and changes produced by drying the aggregate.

With respect to plant control during construction, samples of the mixture must

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be taken constantly, specimens prepared and tested, and the results, immediately analyzed if positive action to assure the production of a uniform mix of good quality is taken. Samples sent to a central laboratory for test are suitable only for record purposes and serve no other function. Also, the practical aspects of the construction control problem demand that the test procedure and equipment be simple to operate and enable test results to be obtained quickly. The test procedures which have been developed and presented in the preceding paper involve sampling of the hot asphalt mixture from trucks as they leave the plant, the compaction of several test specimens in molds, cooling to 140 F. for a brief period, and then testing for stability and flow. By this method results can be made available within less than an hour after sampling. It is believed that the ability to obtain and test large numbers of test specimens with the results immediately available is of prime importance. Complicated procedures that limit the number of tests that can be secured and in which the results are long delayed have no place in the stress of a construction job. The method of design and control has been put to actual use during the construction of several paving projects at the Waterways Experiment Station. Satisfactory blends of local aggregates have been developed, optimum asphalt content determined, and the production of the asphalt plant closely controlled by a systematic sampling and testing schedule. The discussion presented in this paper is based on practical experience obtained during actual construction operations and not on theoretical hypotheses.

A brief reference was made above to certain factors that tend to modify the preliminary design mix. No attempt should be made to prepare test specimens for use as the final design until the plant has been properly adjusted to produce the desired aggregate gradation. Since one or more of the aggregate sources may contain quantities of material that vary considerably in gradation, every effort should be made to improve the uniformity of the gradation of the material by proper construction methods. The careful placement of large aggregate stock piles at the site of the work which permit thorough mixing of material is desirable. If two or more aggregates are being used, the proportion of each to produce the desired blend must be adjusted carefully. Finally the efficiency of the screens which separate the material into the different bins must be determined so that the proper amount of aggregate can be taken from each to meet the desired job mix gradation curve and also maintain a proportionate balance in the different aggregate bins. This may be quite difficult if the screens tend to choke up or if there are marked differences in the moisture content of the aggregate from time to time. Obviously a job mix formula should utilize the entire amount of aggregate available if a satisfactory mix can be produced. Until these adjustments are completed a few specimens can be prepared and tested as an aid to selecting a tentative approximate asphalt content during the period of plant adjustment.

With the plant adjustments made with reasonable accuracy, several batches should be run through the plant at about four asphalt contents intended to bracket the optimum amount. A sample of each mixture should be taken and eight specimens should be prepared at each asphalt content. The test properties for each mix should be determined and the values for each plotted so that curves can be drawn through the plotted points. Optimum asphalt can be determined and the values for stability, flow, percent voids total mix, and percent voids filled with asphalt can be compared with the design criteria. Depending on the results of this analysis, certain decisions are made and may include only a revision of asphalt content or some slight adjustment in the bin proportions. At the time samples of the asphalt mixture are taken to determine the final design, samples should also be taken of the aggregates from the several plant bins for sieve analysis. From a sieve analysis of the material in each bin, and knowing the amount from each weighed into the

hopper, a gradation curve for the combined aggregate can be computed. It is important to know the exact amounts of coarse and fine aggregate, since often they may be of different type and have different specific gravity values. Since values for specific gravity are used in the formulas to compute the density relationships, the exact proportion of each type aggregate used in the mix should be determined. Also, in the event that the trial mix proves unsatisfactory and fails to meet the design criteria, a study of the combined aggregate gradation curve will usually indicate the proper steps necessary to correct the aggregate to produce a satisfactory mix.

With the mix design established, check specimens should be made at regular intervals. These should be tested for stability and flow, and occasionally the unit weight and void relationships can be computed. Reasonable latitude should be tolerated in test results, as some variations in the values can be normally expected. However, if there is a definite trend for, say, the flow to be high, then a study of the aggregate gradation may give a clue as to the trouble. It is seldom necessary to rerun a complete curve of test properties versus asphalt content, even though considerable changes are subsequently made in aggregate gradation from the preliminary mix design. If at optimum asphalt the flow was 15 for a given aggregate gradation, minor changes in gradation may require a different amount of asphalt. At optimum, however, the flow still will be about 15. In this respect it should be noted that the design criteria indicate a maximum flow of 20 for a satisfactory mix. These criteria are based on the results of the traffic tests previously discussed. Practically every asphalt paving item with a flow value not exceeding 20 was satisfactory, and, in general, those with flow values in excess of 20 were unsatisfactory. It is believed that if for a given type and gradation of aggregate the flow value at optimum is, say, 15 and if subsequent tests indicate the flow value has increased to 20, it is probable that the mix would still be entirely stable. It is reasonable to assume, however, that optimum asphalt has been exceeded and the asphalt content probably should be reduced until the flow is again about 15. By the same method of reasoning it is possible that a mix which has a flow of 20 at optimum asphalt still may be satisfactory if occasionally the flow increases to 25.

Experience has indicated (as explained in Paper No. 5) that it is possible with good construction rolling to place the pavement at a density equal to 98 percent of that secured by the laboratory compaction effort (50 blows of a 10-lb. hammer with 3-7/8 in. face falling 18 in. applied to each face of the specimen). Thus, if test samples weigh 140 lbs. per cu. ft., the completed pavement should be compacted to a minimum density of 137 lbs.

It should be emphasized that good construction control can be secured only by providing proper facilities for making the required tests, competent and sufficient personnel to do the work, and prompt recording of the results to permit an early analysis to be made of the results obtained. A systematic laboratory program insures a high quality finished pavement and eliminates the necessity for "guesses" to be made by the engineer in charge.

DESIGN OF ASPHALT MIXES AS RELATED TO OTHER FEATURES OF FLEXIBLE PAVEMENT DESIGN

by
W. J. Turnbull*

A flexible pavement consists generally of the subgrade, one or more layers of stress-distributing media, termed base, and a bituminous weather resistant layer, termed the pavement. Many variations of the generalized section are to be found. For instance, the base may be of bituminous construction; also, the pavement may be a part of the stress-distributing system.

It is not necessary to elaborate on the basic principles of a flexible pavement. These have been given by Benkelman *(4)*, Spangler *(5)*, Porter *(6)*, Boyd *(7)*, and many others. Most engineers reach agreement on the basic principles of design by stating that sufficient stress-distributing media should be provided between the wheel load and the subgrade so that the stresses induced in the subgrade do not exceed the supporting strength. While practically complete agreement has been achieved on the basic functional principles of a flexible pavement, such agreement has not been reached on the method of design which fulfills this, and other necessary requirements. It is not the purpose of this paper to try to achieve agreement on a method. Rather it is the purpose to review briefly what the Corps of Engineers considers desirable in a flexible pavement, the methods that have been used in designing its various components, and how the results of the study presented in the preceding papers of this symposium will aid in future flexible pavement design.

In designing flexible pavements for airfields to be constructed in the United States by the Corps of Engineers it is considered that the following should be provided to insure long life with little or no maintenance for the design wheel loads.

(a) Adequate total thickness of base and pavement to prevent shear deformation in the subgrade.

(b) A combination of base course strength and pavement thickness so that no shear deformation occurs in the base.

(c) Sufficient compaction in the component layers to prevent detrimental settlement under traffic.

(d) Adequate quality in the pavement to resist displacement under hot weather traffic, to resist cracking and disintegration during cold weather traffic, and to form a weather and abrasion resistant medium.

As is well known, items (a) and (b) above are designed by the California Bearing Ratio Method (CBR) as revised by the Corps of Engineers (except where frost is a factor). The Corps of Engineers has completed a considerable volume of research work on the CBR test, the preparation of laboratory specimens to an assumed design condition, and the validation of the design curves. This work has been reported in a symposium entitled "Development of the California Bearing Ratio Method of Flexible Pavement Design for Airfield Pavements," which has been submitted to the American Society of Civil Engineers for publication in the ASCE Proceedings. The asphalt stability test section described in a preceding paper furnished information on pavement thickness requirements to protect any specified underlying base course. It is

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believed that the CBR design curves are valid and that they reflect with reasonable accuracy the total thickness of base and pavement required above a soil with a given CBR value. Also, means have been developed for evaluating the effect of multiple wheels and for using the CBR curves in designing for multiple wheels. The most severe criticism leveled against the CBR system is the method of preparing a laboratory specimen to the assumed design condition that is, the soaked condition. It is admitted that the method has not been perfected to the point where one can be assured that the specimen tested in the laboratory will be duplicated in the prototype; however, this criticism is not unique to the CBR method. Any design procedure must utilize tests on specimens prepared in advance of construction and the problem of preparing specimens to meet a given design condition will always be present. Test specimens may be prepared in the laboratory and the moisture content appropriately adjusted, or the design must be based on the results of actual field tests. In the latter case, the problem of adjustment for future moisture conditions is intensified. The Corps of Engineers is continuing its study to develop a method of preparing specimens to simulate the prototype for a wide range of climatic and drainage conditions. The method now being used provides a design which will be adequate under the most adverse soaked conditions; at any time when the subgrade condition is less severe than this "adverse condition" the design will be conservative. Further, the method can be delineated and followed with consistency by a wide group of engineers, a feature which is desirable where a construction program blankets the entire United States.

The procedure used by the Corps of Engineers to provide sufficient compaction in the component layers to prevent detrimental settlement is to specify given percentages of Modified AASHO density in each layer. The percentages that are specified have been established on the basis of measurements of the compaction produced in bases and subgrades by accelerated traffic tests and by actual airplane traffic. High degrees of compaction are specified near the surface for heavy wheel loads; these are reduced both as the depth increases and the load decreases *(8)*. It is considered that such criteria are adequate to insure that detrimental settlement will not occur under future traffic.

The criteria that have been used by the Corps of Engineers for insuring adequate quality in the asphalt paving mixtures have been primarily based on experience. Rather strict requirements have been maintained for materials and gradation. The final proportioning of the ingredients was left mainly to the judgement of the engineer designing the mix, who was free to use any type of test equipment for evaluating the selected paving mixture. The design of the mix was still pretty much an "art" and in most instances the results obtained depended largely on the experience of the designer. The establishment of the Flexible Pavement Laboratory by the Office, Chief of Engineers, and the early assignment of the problem of developing a method of design and control of bituminous mixtures are evidence that the need was recognized.

It is desired to stress that the study described by this symposium shows that the asphalt content is the most important variable in an asphalt paving mixture. In the field test section a wide range existed in a number of variables, such as aggregate type, aggregate gradation, and filler content. By comparison the asphalt content for any one blend was varied only 20 percent. For example, one of the asphaltic concrete mix series was placed at 6.6, 6.0, and 5.4 percent asphalt. Almost without exception the rather narrow numerical range in asphalt content as illustrated by the above example was more important in determining whether mixtures were satisfactory or not than any other feature.

Experience indicates that the density of a paving mixture tends to increase under the kneading action of traffic. Pavements whose asphalt contents are considered satisfactory at the time of con-

struction may flush and become unstable under traffic due to increase in density. Designs must be based on specimens which are compacted to a degree approaching that which they will reach in the pavement after they have been subjected to considerable traffic. The design method presented in this symposium presents a laboratory compaction procedure which has been correlated to produce the degree of density achieved by the traffic in the test section as described in Paper No. 4. Correlation based on prototype pavements has generally shown that if good construction practices are followed, a pavement should be compacted at placement to about 98 percent of the design density.

Numerous engineers have used rolling specifications which require a compaction of 90 to 95 percent theoretical density without realizing that they were neither specifying a definite amount of compaction nor obtaining sufficient compaction in many cases. It is quite easy to roll to 95 percent theoretical density if the mix has a little extra asphalt in it, but there is no assurance that additional compaction will not occur during traffic and cause the pavement to flush or become unstable. It is believed that this type of specifications should be revised to take advantage of the information developed in this study.

The procedures and criteria developed in this study will permit the engineer to select, with confidence, the maximum or optimum amount of asphalt that can be included in any given blend without producing a mixture that will displace under warm weather traffic. Further, he can be assured that the mixture can be controlled during construction so that when built it will have the characteristics required by the design. A design for the maximum allowable amount of asphalt automatically insures maximum durability and life for that particular blend. Additional work is being carried on with respect to the flexibility or durability characteristics of asphalt mixtures for the purpose of developing some test or criteria by which the flexibility can be controlled by specific design. In the meantime, it is believed that the design of asphaltic mixtures with the greatest amount of asphalt commensurate with anticipated airfield traffic will be an adequate "stop-gap" for insuring flexibility. The design method which has been developed accomplishes the objective set forth in the preceding sentence.

With a method established for selecting the optimum asphalt content and for evaluating the quality of a paving mixture, the engineer can make logical comparison between the test properties of various blends and select the best aggregates from both the technical and economic viewpoints. It is fully expected that test properties of heretofore rejected local aggregates may often prove to be as satisfactory as those from imported aggregates.

The real value of the method is best illustrated by assuming that a given aggregate material must be used in an asphaltic pavement construction job in a forward theater of operation, without benefit of any adjustments in gradation or filler content. For this condition the method furnishes a reasonable basis for arriving at the proper asphalt content which will produce the best possible asphaltic mixture utilizing the materials available. Many engineers have felt the need of a method of asphaltic mixture design which has a more scientific basis that the presently used "art" basis. The method of design discussed in this symposium supplies this need in the sense that it is based on logical procedures.

It is pointed out that stability is only one test property out of five used in the procedure. Stability is used primarily as a tool for selecting asphalt content and its worth in this respect has been well demonstrated. It is admitted that so far stability alone is not satisfactory for evaluating an asphaltic mix. As pointed out in a preceding paper, the curve of stability versus asphalt content peaks so that two equal stability values can be obtained, one on the lean and one on the rich side of optimum. This feature would be true of any other type of stability test which develops a maximum value with changes in asphalt content. One highly

desirable feature of the Marshall specimen is that it can be cut from a completed pavement and tested without reshaping or remolding. It is considered that any test must have this feature to establish a satisfactory means for laboratory and field correlation. Another favorable festure of the Marshall test apparatus is its light weight and flexibility of use. The apparatus can be used in the laboratory or field for design, construction control, or evaluation purposes; also, it can be manually or mechanically operated.

It is to be pointed out that the Marshall apparatus measures empirically certain properties of asphalt pavements. However, in this investigation thicknesses and properties of asphalt pavements suitable for a range of wheel loads were established as the result of submitting various sections of design asphalt pavements to actual traffic. It is self-evident that any theoretical test apparatus or test method used for the design of asphalt pavements should be validated by traffic. This is particularly true where the apparatus is to be used for designing pavements suitable for wheel loads of the magnitude for which little or no engineering data are available. The basic principle of compacting test specimens to the density which they will achieve in the prototype as utilized in the method proposed in this symposium is considered sound and logical. Total base and pavement thickness and compaction requirements are designed by empirical methods by the Corps of Engineers; consequently there is nothing illogical in the development and use of the pavement design method presented in this symposium. The Corps of Engineers has long been interested in a more rational method of design for all features of a flexible pavement and is continuing large-scale stress distribution studies to measure the distribution of stresses and strains under simulated airplane wheel loads. The triaxial test is being investigated, including repetitive loading, for measuring stress-strain characteristics. The work has progressed to the point where it is apparent that the development of a rational design method for total thickness of base and pavement based on quality of subgrade, base course and wearing course will be a major undertaking and will require much more work and time to consumate.

At this time it is desired to point out that the design and control procedures for asphaltic mixtures proposed by the Corps of Engineers use a refinement of the Original Marshall equipment for the stability and flow tests. Mr. Marshall devised the stability test and equipment while with the Mississippi State Highway Department. During his association with the Flexible Pavement Laboratory the flow test was developed. The compacting and test procedures, the design criteria, and refinements to the basic equipment have been developed since Mr. Marshall left the Flexible Pavement Laboratory. The Corps of Engineers expresses appreciation to Mr. Marshall for his contribution in the development of the original stability test and apparatus.

In summation, it is considered that the procedures and criteria proposed in this symposium for the design and control of asphalt paving mixtures within the scope studied will permit the engineer to design asphalt pavements with confidence. These procedures and criteria, together with the CBR method of thickness design and the compaction requirements, round out the Corps of Engineers' present flexible pavement design plan.

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GLOSSARY OF TERMS

A proper understanding of the terms defined below will be of assistance to the reader in the study of this Symposium.

GENERAL TERMS

Asphalt paving mixture - The combination of aggregates, sand, filler, and asphalt cement properly proportioned to produce the desired mix.

Flexible pavement - This term is used when reference is made to the entire section consisting of one or more stress distributing layers known as base and subbase courses and the pavement.

Pavement - The bituminous mixture superimposed on the base as an abrasion and weather resistant structural medium.

Wearing course - The top course or wearing surface of the pavement.

Binder course - The lower course of the pavement immediately above the base. This term is used only where the pavement consists of more than one course.

Bituminous surface treatment - A treatment of hot bitumen applied directly to a prepared base followed by an application of aggregate. Additional applications of bitumen and aggregate may be applied to secure any desired thickness.

Asphaltic concrete - A type of pavement composed of a mixture of coarse aggregate, fine aggregate, filler, and asphalt cement containing approximately 40 percent by weight of aggregate retained on the No. 10 sieve or in sufficient proportion that interlocking of the coarse aggregate occurs. The exact amount of coarse aggregate that produces interlocking is dependent upon size, specific gravity, and character of the aggregate.

Sand asphalt - A type of pavement composed of a mixture of fine aggregate, filler, and asphalt cement.

Stone filled sand asphalt - A type of pavement composed of a mixture of coarse aggregate, fine aggregate, filler, and asphalt cement containing aggregate retained on the No. 10 sieve in quantities generally less than 40 percent by weight or such that will preclude interlocking of the particles. The exact amount of coarse aggregate is dependent upon size, specific gravity, and character of the coarse aggregate.

Coarse aggregate - The material retained on the No. 10 sieve.

Fine aggregate - The material passing the No. 10 sieve and retained on the No. 200 sieve.

Filler - Mineral aggregate particles passing a No. 200 sieve.

Crushed limestone - A limestone found

in a solid state, quarried, and crushed to meet the grading requirements established for coarse aggregate in the investigation.

Crushed gravel - Rounded or partially rounded gravel which has been crushed and screened to meet the grading requirements established for coarse aggregate in this investigation.

Slag - A non-metallic product consisting essentially of silicates and calcium compounds, a by-product of the steel industry. The product when cold was crushed and graded to meet the requirements established for coarse aggregate in this investigation.

Gap-graded aggregates - An aggregate having a complete void in one or more fractional grain sizes.

Asphalt cement - A fluxed or unfluxed petroleum asphalt prepared to a certain quality and consistency for direct use in the manufacture of asphalt pavements.

Modified AASHO hammer - A compaction device in which the hammer weighs 10 lb. and falls through a distance of 18 in. upon a foot 1.95 in. in diameter.

TERMS RELATING TO TEST PROPERTIES

Stability - The ability of the pavement to resist the shearing stresses imposed by the wheel loads without displacement.

Marshall Stability value - The maximum load in pounds required to produce shear failure in a compacted specimen of the asphalt paving mixture when tested in the Marshall apparatus.

Flow value - The total deformation, measured in hundredths of an inch, that occurs in the compacted specimen of the paving mixture at the point of maximum load when subjected to the Marshall Stability test.

Unit weight total mix - The total weight of an asphaltic mixture, including all aggregates and asphalt, in lb. per cu. ft. synonymous with *total weight*.

Unit weight aggregate only - The total weight of all aggregate in an asphaltic mixture in lb. per cu. ft. The term is synonymous with aggregate weight.

Percent voids aggregate only - The total percentage of voids in the compacted aggregate mass including those occupied by asphalt and air. The term is synonymous with *aggregate voids* and *percent voids in the mineral aggregate* and is the complement of *percent solids aggregate only*.

Percent voids total mix - That part of the compacted asphalt mixture not occupied by aggregate or asphalt expressed in percent of the total volume. It is synonymous with *air voids* and is the complement of *percent solids total mix* and *percent of theoretical maximum density*.

Theoretical maximum density - The theoretical specific gravity of the total asphalt mixture in which it is assumed that all voids are eliminated. If expressed in pounds, the value is multiplied by 62.4

Percent voids filled with asphalt - The percentage of the voids in the compacted aggregate mass which are filled with asphalt cement. It is synonymous with *asphalt void ratio*.

Optimum asphalt - That asphalt content in the pavement mixture which is judged to be most desirable by one or more criteria.

TERMS RELATING TO THE TEST SECTION

Test section - The entire area consisting of tracks and turnarounds constructed for the traffic tests.

Test track - The straight portion of the test section upon which the traffic tests were conducted. There were two parallel test tracks, each having three lanes and a variety of types of pavements.

Turnarounds - Areas at each end of the test track used for turning and maneuvering the traffic equipment. The turnarounds were composed of various types of pavements used in special studies.

Lane - The area of the test track subjected to controlled traffic. Three lanes were designated on each track for the 15,000-, 37,000-, and 60,000-lb. wheel loads respectively.

Section - A major subdivision of the test section in which the aggregate used in the pavement is of one type and placed on one quality of base.

Subsection - A division of a section in which the pavement is of constant thick-

ness. (In this symposium the subsection designation is combined with the section number and the combined symbol used to designate a section of the test track.)

Unit - A division of a subsection in which the aggregate gradation is uniform but the asphalt content may vary.

Item - The smallest subdivision of a test lane in which all factors are constant.

Coverage - A single wheel application over all points in a given traffic lane.

DISCUSSION

V. R. SMITH, *California Research Corporation* - The work undertaken by the Army Engineers and the data presented in this symposium, (including the reports frequently referred to) probably represents the most extensive study of asphalt paving mixtures yet undertaken. A great deal can be said concerning the merits of this work and much less concerning its faults. These comments are intended as constructive criticism in the interest of developing better methods for the design and control of asphalt paving mixtures which is the purpose of this symposium.

Selection of the Marshall Test - It was emphasized in the papers by Messrs. Mc Fadden and Ricketts and by Mr. Griffith that for military reasons a test method of utmost simplicity and portability is desired. The Marshall test meets these requirements. However, in achieving these ends the Marshall test procedure sacrifices an extremely important property; namely, it does not give due weight to the frictional resistance properties of bituminous mixes. These frictional resistance properties are fully as important as the tensile strength characteristics which are the predominant influence in the Marshall test. Messrs. McFadden and Ricketts stated "that the Hubbard-Field machine was typical of those that satisfactorily measured the pertinent properties of an asphalt paving mixture." Because the Marshall test gave good correlation with Hubbard-Field results, it was chosen as the basic laboratory tool for all subsequent studies. There is a large and rapidly growing group of highway engineers that has become convinced that the Hubbard-Field test does not measure all the "pertinent properties of an asphalt paving mixture" but instead is primarily affected by the tensile strength or cohesion properties of a mixture with frictional resistance characteristics influencing the test results to a minor degree. Other tests which fall in the same category are the unconfined compression test, the several extant punching shear tests, direct shear tests, and the Hveem Cohesiometer. All of these tests measure tensile strength (cohesion) predominantly. Although they are valuable tools in establishing the cohesive properties of a bituminous mix, they provide only limited indications of the frictional resistance properties, the latter being of paramount importance in withstanding compressive stresses.

We have all learned that any test method which has been widely used and with which we are personally experienced is a valuable tool for guiding our thinking when new and untried situations develop. On the other hand, we are inclined to allow our experience and understanding to influence the meaning of numerical test results obtained from an emperical test. Thus a given numerical test result does not always mean the same nor is it handled the same by the neophyte as by the experienced engineer. The Marshall test appears to suffer from this shortcoming as some engineers now becoming acquainted with the test are having difficulty in interpreting the test results in the light of their field experience. It is the writer's opinion that the Marshall test must be complemented by a test which measures frictional resistance properties before the former can be widely employed. Other than the fact that the Marshall test can readily handle cored specimens of any reasonable thickness, it does not offer any advantages, nor does it provide any information, which cannot be developed from a simple unconfined compression

test.

Shortcomings of Marshall Test - This test does not adequately measure frictional resistance toward compressive stresses primarily because the specimen is unconfined on two sides. The test is made at a rate of strain which is excessive in that it will not indicate the true cohesive properties of a mixture subjected to static loads, which are recognized as the most severe stress conditions encountered. The fact that eight "identical" specimens are prepared for each test indicates that the test does not possess as high a degree of reproducibility or preciseness as would be desirable.

The accompanying Figure 1 illustrates the effects of asphalt content upon friction and cohesion (interlock plus tensile) properties of a typical well-graded asphaltic concrete mixture. These particular data were obtained from triaxial tests using a closed system triaxial cell. Load was applied in static increments. However, it is of special significance that the "cohesion" curve has exactly the same shape and peaks very close to the same asphalt content as do "stability" vs. asphalt content curves derived from Marshall tests, unconfined compression tests, Hubbard-Field tests and similar tests which measure predominantly the cohesive or tensile properties of a mix. It will be observed that, for this particular aggregate, at the asphalt content which is optimum for "cohesion," the frictional resistance properties of the aggregate have been seriously reduced due to lubrication by the bituminous binder. The calculated unit vertical pressures which these mixes will withstand without undergoing plastic shear deformation are shown. Whether these calculated values are correct or not, it is still significant that internal friction is often greatly reduced at the asphalt content which is optimum for "cohesion," or optimum by Marshall test, or optimum by Hubbard-Field. Thus mix design by Marshall test, Hubbard-Field, unconfined compression, or like test will often yield pavements on the "rich side" and likely will result in shoving or rutting distress.

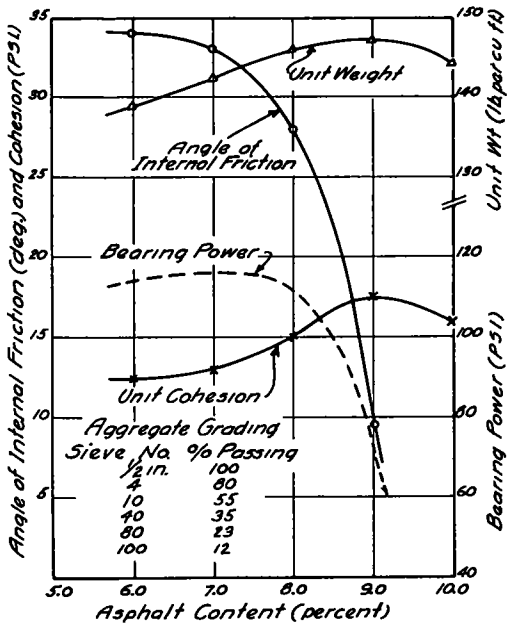

Figure A.

It has been found that certain mixes are less critical of asphalt content than the mix shown in Figure 1. In these cases, internal friction remains at a relatively high value at "optimum cohesion." In such cases maximum stability will occur at or near the Marshall optimum asphalt content. However, many mixes (particularly the dense, uncrushed, high surface area type) will be reduced in internal friction on increase of binder content even more rapidly than the mix of Figure 1. In these cases, design by Marshall Test is likely to yield a very unstable overly rich mix. The asphaltic concrete mixes employed in the field studies reported by Mr. Foster have been tested for both internal friction and "cohesion" at various asphalt contents and have been found less critical of binder content than the mix represented by Figure 1. Had "critical mixes" been included in the field studies, poorer correlation with laboratory test results would have been expected.

While frictional resistance properties are being discussed, mention should be made of the results obtained on testing

road-mix asphaltic surfaces by the Marshall test or similar tests. Many road mixes made with the lighter liquid paving asphalts such as SC-2 or MC-2 fall far short of meeting the Marshall stability design criteria of 500 + discussed in Mr. Griffith's second paper of this symposium. However, these mixes are exhibiting ample load carrying capacity in many locations of severe traffic. The reason such mixes often perform so satisfactorily in spite of their low tensile strength as measured by Marshall test, Hubbard-Field, or unconfined compression test is because they possess relatively high internal friction properties. Many road oil mixtures which possess extremely low unconfined compression strength, low Marshall strength, and low Hubbard-Field strength, but which are giving completely adequate service performance have been investigated. It has been found without exception, when these mixes are tested for triaxial stability, that they possess excellent frictional resistance properties although very low in "cohesion."

A further indication that design based on Marshall test results may sometimes yield erroneous conclusions is found in Figure No. 9 of Mr. Boyd's first paper. In this figure it is shown that an aggregate with size distribution approximating aggregate number 10 shown in the same paper, Figure 5 (76 percent coarse aggregate with 3/4 inch maximum, and 24 percent minus 10 mesh with about 4 percent 200 mesh fines) possesses a very low Marshall Stability at Marshall optimum asphalt content. Yet asphaltic concrete mixes of precisely this grading were adopted in Pacific Coast shipyards and subsequently for extensive airport and highway construction in the Pacific Northwest. In the case of the "shipyard pavements," it was necessary to find asphaltic concrete mixes which would withstand unit surface loads several times greater than are applied by the heaviest aircraft in order to withstand the effects of armor plates standing on edge, and similar loads. Crushed rock (3/4 inch maximum) graded precisely according to curve No. 10, Figure 5, of Mr. Boyd's paper was found by trial and error to possess adequate stability to withstand these excessive contact pressures, yet such a mix shows a Marshall test stability of only about 250 lbs. In the "shipyard pavements" an asphalt content of about 5.1 percent is used, a mix containing about 16 percent voids results, and an asphaltic binder of 200/300 penetration or harder is satisfactory.

Mr. Boyd's first paper states, "Increased Marshall stability can be secured by using a lower penetration asphalt." This assertion is supported by Figure 11 of the same paper. The tremendous increase in Marshall stability derived by using harder asphalts (280 lbs. stability with 120 penetration binder as compared to 440 lbs. with 54 penetration binder) has not been observed in actual service within the scope of the writer's experience. Instead it has been found that a mix employing 120 penetration binder which shoved and corrugated at boulevard stops acted virtually the same when a binder of less than 50 penetration was employed. Similarly a desert sand mix which ruts when bound with SC-2 or SC-3 also ruts when bound with 200/300 penetration asphalt. The pronounced effect of asphalt grade on Marshall stability is due to the relatively high rate of strain (2 in. per minute) employed in making the test. A mix showing 500 lbs. Marshall stability will probably show less than half that stability if tested at 0.2 in. per minute. If tested at 0.002 in. per minute its stability may well be less than 20 pounds. Likewise the changes in stability resulting from changes in penetration grade of binder as illustrated in Figure 11 of Mr. Boyd's first paper will disappear to a large extent if testing is conducted at a much lower rate of strain or at near static conditions. The reason for this can be traced to the viscous nature of asphalt and/or asphalt-filler mixtures. Fifty penetration asphalt offers an exceedingly high resistance when deformed at a noticeable rate, whereas it offers virtually no resistance to deformation at extremely low rates of strain. Indeed we all have seen a chunk of 50 penetration asphalt flow out into

a puddle when left at room temperature for several weeks. The rate of strain employed in the Marshall test lies between these two extremes. For this reason the resistance offered to deformation in the Marshall apparatus approximates the cohesive resistance which the mix will offer to moving wheel loads. It has little relation to the resistance which the mix will offer to static loads (parked aircraft, parked automobiles) which have been proven to represent the most severe loading condition insofar as an asphalt mixture is concerned. Therefore, good correlation between Marshall test results and service behavior of taxiways or city pavements should not be expected.

The need for using eight test specimens to assure adequately accurate test data is discussed in Mr. Griffith's second paper. The fact that such a large number of specimens of a specific mix are required indicates that the reproducibility of the test is not as high as might be desired for simplicity, economy, and expeditiousness. It is the writer's belief that this poor reproducibility is due mainly to the effects of temperature on the viscosity of the asphalt binder, and the effects of oxidation of the asphalt during preparation of the specimens. Figure 11 of Mr. Boyd's first paper shows that a reduction of several points in asphalt penetration results in a sizeable increase in Marshall stability. A difference of several points in the penetration of the asphalt binder will often result even though mixes are prepared strictly according to the very specific procedures and temperature tolerances outlined in the test method. Similarly slight differences in the actual temperature of the briquet being tested can result with changes in room temperature, use of cold testing heads as compared to testing heads that have become warm due to contact with previous specimens, etc. Any slight change in the temperature of the specimen will greatly affect the viscosity of the asphaltic binder and will be reflected in erratic stability results when testing is conducted at a high rate of strain. It is suggested that the Army Engineers further explore the effects of very low testing speeds in order to improve test reproducibility.

Army Engineers Proposed Design Criteria - In Mr. Griffith's second paper very specific design criteria are listed for establishing the suitability of a proposed paving mix. For asphaltic concrete these criteria are given as:

Marshall Stability	- more than 500
Flow	- less than 20
Percent Voids, Total Mix	- 3 to 5
Percent Voids filled with Asphalt	- 75 to 85

The need for a Marshall stability of over 500 to satisfactorily carry heavy aircraft or other traffic has been questioned above. Field observations have demonstrated that many road-mixes of extremely low Marshall stability or unconfined compressive strength perform quite satisfactorily because they possess high frictional resistance characteristics, a property which does not influence the Marshall test results to the proper degree.

The "flow test" introduces a new empirical measure of the plasticity characteristics of bituminous mixtures. Undoubtedly, it measures the same property as is obtained from an unconfined compression test when the percent strain at maximum load is measured. The writer has been unable to apply the results of such tests to any specific characteristic observed in the field. However, this property may be useful in wearing surface design. It will be quite interesting to determine how certain road-mixes, asphalt macadams, and high voids content mixes react to this determination.

The limitation percent voids total mix of 3 percent to 5 percent will probably be questioned in many quarters. Samples of unstable wearing-courses taken from various locations on the Pacific Coast have shown percent voids total mix of 4 percent or less. This is in substantial agreement with the Vicksburg findings that instability results at percent voids total mix of 3 percent or less. The measured voids content is subject to some

variations depending upon the methods used for measuring specific gravity of the constituents and other factors. There is no apparent justification for a 5 percent maximum percent voids total mix, as many surfacing mixes having voids contents considerably in excess of 5 percent are being used today after many years' satisfactory service. Extremely high voids content mixes (above 10 percent) are likely to leak water and therefore must be used over foundations relatively unaffected by moisture or they must be provided with a seal coat. However, from a stability standpoint and from a flexibility standpoint (lack of brittleness) there is no reason for establishing 5 percent voids total mix as the maximum allowable. The brittleness characteristics of an asphaltic mixture are dependent upon the thickness of the binder films on the surfaces of the mineral particles which is only indirectly related to the voids content of the compacted mix. In support of this argument, the accompanying Table 1 is presented. The so-called "open-graded" mix of Table 1 represents a somewhat radical type of asphaltic concrete and one which might leak water if not sealed. However, from a stability standpoint this type of mix has demonstrated the ability to withstand surface stresses greatly in excess of those imposed by the heaviest aircraft. Likewise this type of mix has demonstrated the capacity to deform over 1 inch in a radius of 12 inches without visible signs of cracking or loss of soundness. Yet field specimens of this surfacing show voids contents of 16 percent or slightly more. Of course, this flexibility results from the relatively thick films of binder which can be employed with this aggregate gradation without introducing excessive lubrication. Mix No. 11 of the Vicksburg experiments represents a more "normal type" of asphaltic concrete. Comparing it with the open-graded mix of Table 1, it is apparent that on a surface area basis the binder films in the high voids content mix are 65-85 percent thicker than those of Mix No. 11. Many similar cases can be cited which substantiate the contention that 5 percent voids total mix is entirely unreasonable as a design criterion. In fact, it appears that voids content per se has little relation to the suitability of an asphaltic mixture. If this is correct, the proposed limitation of 75 percent to 85 percent voids filled with asphalt is also irrevelent.

Preparation of Marshall Test Specimens - Mr. Foster's paper emphasizes that compactive effort has a considerable influence upon the measured stability properties of an asphaltic mix. He states that "if values are to be established for the test properties, they must be made with respect to a very definite compactive effort which must be specified and used to qualify such values." This matter deserves maximum emphasis. In support of the Army Engineers' findings the data of Table 2 are presented. These data illustrate the differences in measured stability values resulting from different types of compaction. Mix A contains 7.0 percent binder and its measured stability properties are little affected by type of compaction. Mix B and Mix C, the same aggregate with 8.0 percent binder exhibit widely different stability properties depending upon the method of compaction employed. In this case, double plunger compaction gives a much higher stability at a given mix density than does the mechanical (kneading type) compaction. The reason for this difference lies in the fact that extreme pressures (2000 psi or more) must be employed in plunger compaction to achieve the densities that are obtained with a kneading action or with a roller in the field at relative low compaction pressures (200-400 psi). These extreme pressures result in puncturing of binder films between particles, films which are not punctured but continue to act as lubricants under field compaction conditions. Also these extreme pressures result in excessive fracturing of the aggregate particles. It is not known how well the impact method recommended for preparing specimens in Mr. Shockley's paper will approximate the particle orientation obtained during field rolling. However, since a compaction foot covering

TABLE A

COMPARISON OF PROPERTIES OF ASPHALTIC CONCRETE MIXES

Size Distribution	Vicksburg Mix 11	Pacific Northwest Open-Graded Mix (Shipyard Mix)
Passing 3/4 in. sieve, %	100	100
Passing 1/2 in. sieve, %	95	77
Passing 3/8 in. sieve, %	73	67
Passing No. 4 mesh sieve, %	58	40
Passing No. 10 mesh sieve, %	52	21
Passing No. 20 mesh sieve, %	46	17
Passing No. 40 mesh sieve, %	32	11
Passing No. 80 mesh sieve, %	13	6
Passing No. 200 mesh, sieve, %	6	4
Asphaltic Binder, %	5.4-6.0[a]	5.1
Voids Total mix, approximate, %	2-7	16
Approximate Surface Area, sq. ft. per lb.[b]	37.3	19.2
Thickness of Asphaltic Binder Films, microns	6.2-6.9	11.4

[a]Considered optimum range.
[b]According to California Division of Highways Method.

the entire specimen is employed, it would appear that little opportunity is provided for the aggregate particles to orient themselves as they will under rolling or kneading compaction. In other words, it is likely that some critical mixes, such as Mix B or Mix C of Table 2, will give high Marshall stabilities and yet exhibit shoving distress in service. Likewise, it would be expected that such critical mixes will yield Marshall stabilities on laboratory specimens greatly in excess of the stabilities obtained on cored specimens.

The data of Table 2 together with voluminous additional data from a variety of sources prove that density, per se, has little meaning in the design and testing of bituminous mixtures. It is the method employed in achieving density which is most important insofar as the load carrying properties of the resulting mix are concerned. This matter certainly deserves further study and such studies now are being undertaken by various interested groups.

Vicksburg Traffic Tests and Evaluations - The construction and testing of the traffic sections at Vicksburg undoubtedly represented a tremendously difficult problem because of the ramifications involved and the completeness and accuracy desired. All who have had experience in the construction and testing of field sections realize the problems involved and will praise the manner in which field tests were conducted by the Waterways Experiment Station and the thoroughness and preciseness achieved. It is difficult to assimilate and evaluate quickly all of the data presented in Mr. Foster's paper and the Waterways Experiment Station reports to which he refers. Certain test results and conclusions highlight themselves when they serve to confirm or to contradict one's experience or other laboratory or field test results. It is those conclusions which do not seem to be

supported by the actual data and those which seem to contradict previous experience that are discussed below.

In order to eliminate the possibility that surface distress resulted from improper foundation support these remarks are confined to results observed on sections 1A, 1B, 1C, 2A, 2B, and 2C which consisted of asphaltic concrete mixtures placed on thick, high bearing power bases (80 + CBR) on a strong subgrade (20 CBR).

In the accompanying Table 3 pertinent data have been tabulated for Mix 12 and Mix 15 which comprised the high Marshall stability (550 lbs.) mixes composed of crushed limestone and uncrushed gravel, respectively. The data of Table 3 were copied from tables C-1, D-1, D-9, D-12, D-13, and D-14 of the Waterways Experiment Station Technical Memorandum No. 3-254, and are typical of the voluminous data which appear in this memorandum.

faint upheaval occurred. The optimum binder content for Mix 15 is reported as 5.0-5.5 percent. The behavior of this mix within this binder range parallels that of Mix 12 within its so-called optimum binder content range.

An appraisal of the suitability of these mixes for the heavy wheel loads applied, depends upon the minimum standard of quality considered essential, Most paving engineers consider that any indication of rutting, shoving, or upheaval within the surfacing mix is indicative of unsatisfactory stability and high maintenance requirements in the future. By these standards neither Mix 12 nor Mix 15 would be considered suitable for heavy aircraft loads or unlimited highway traffic. Incidentally, Mix 12 has been tested by the triaxial stability method and was found to fall somewhat below the stability requirements

TABLE B

EFFECTS OF COMPACTION PROCEDURE ON MEASURED STABILITY

Mix No [a]	Method of Compaction [b]	Asphalt Binder %	Asphalt Binder Grade	Unit Weight Total Mix lb per cu.ft	Voids Total Mix %	Stability Constants [c] ϕ	Stability Constants [c] C	Calculated Supporting Power psi
A	M C	7 0	120/150	146 7	4 6	38 3	12 1	132
	D P	7 0	120/150	146 4	4 7	39 0	14 2	160
B	M C	8 0	60/70	145 8	3 9	10 8	9 0	38
	D P	8 0	60/70	145 7	3 9	39 4	15 7	180
C	M C	8 0	200/300	146 0	3 8	21 6	12 3	72
	D P	8 0	200/300	145 2	4 0	37 4	13 0	137

[a] All mixes prepared from crushed basalt and graded according to middle of limits of Asphalt Institute Specification A-2-b

[b] M C - signifies mechanical compaction according to method presented in "Journal Asphalt Technology," January 1944 D P - signifies double plunger compaction according to Asphalt Institute Specification A-2-b

[c] According to method of test described in Asphalt Institute Specification A-2-b

It will be noted from Table 3 that the 550 Marshall stability expected for mixes 12 and 15 was achieved in most cases during construction of the test sections and this stability was obtained in all cases after traffic compaction of several hundred passes of the wheel loads. From these and other field test results the Army Engineers conclude that for Mix 12 the optimum binder content is 4.8 percent to 5.4 percent. However, it will be noted that faint or well-defined rutting and shoving occurred in many instances at binder contents of 4.8 percent. At 5.4 percent rutting and shoving was more pronounced in general, and in some cases,

employed in many areas (Asphalt Institute Specification A-2-b).

The data of Table 3 show a trend toward poorer stability as thicker asphaltic surfaces are employed. This phenomenon agrees with theoretical principles, in that a mix having insufficient shearing strength to resist imposed stresses should show greater resistance to movement when it is employed in thinner layers.

The data of Table 4 also were copied from the previously mentioned tables of Technical Memorandum 3-254. These data are typical of tests on traffic sections employing asphaltic concretes of different Marshall stabilities. It can be seen

TABLE C

SUMMARY OF VICKSBURG RESULTS WITH HIGH STABILITY ASPHALTIC CONCRETE MIXES ON HIGH SUPPORTING POWER FOUNDATION

Section	Unit	Item	Asphalt Content Wearing Surface %	Thickness (Inches) Wear Surface	Thickness (Inches) Binder Course	Marshall Stability - Lbs As Placed	Marshall Stability - Lbs At 500 ± Coverages	Service Behavior Noted
					Mix 12 - Crushed Limestone[a]			
1A	3	1	4 8	1 5	0	426-590	912-1250	Well defined tire-printing 15M lane, Faint tire-printing 60M lane
		2	5 4	1 5	0	774-1006	1062-1387	Faint rutting and showing 37M and 60M lanes Well defined tire-printing 60M lane
		3	6 0	1 5	0	448-740	737-966	Faint rutting and showing 15M and 60M lanes Well defined tire printing all lanes
1B	3	1	4 8	1 5	1 5	386-558	960-1257	Well defined rutting 15M lane, faint upheaval 60M lane, faint rutting and showing 37 M lane
		2	5 4	1 5	1 5	583-802	1170-1250	Well defined rutting 15 M lane, faint rutting and showing 37M and 60M lanes
		3	6 0	1 5	1 5	610-893	988-1160	Well defined rutting 15 M and 37M lanes, faint rutting showing 60M lane
1C	3	1	4 8	2 0	3 0	543-733	852-1066	Well defined rutting and faint upheaval 37 M lane Faint rutting and showing other lanes
		2	5 4	2 0	3 0	294-476	902-1342	Well defined rutting all lanes, faint upheaval 15M and 37M lanes
		3	6 0	2 0	3 0	702-834	716-1335	Well defined rutting all lanes, faint upheaval 15M and 37M lanes
					Mix 15 - Uncrushed Gravel[b]			
2A	3	1	4 5	1 5	0	319-427	663-973	Faint rutting and showing all lanes
		2	5 0	1 5	0	727-736	1043-1148	Well defined rutting 15M lane, faint rutting and showing 37M lane
		3	5 5	1 5	0	466-535	866-1173	Faint rutting and showing 37M and 60M lanes
2B	3	1	4 5	1 5	1 5	336-462	846-1162	Faint rutting and showing all lanes
		2	5 0	1 5	1 5	484-705	693-1058	Faint rutting and showing all lanes, well defined tire printing 15M lane
		3	5 5	1 5	1 5	652-747	982-1181	Faint rutting and showing all lanes, well defined tire printing 15M lane
2C	3	1	4 5	2 0	3 0	302-489	737-1072	Faint rutting and showing all lanes
		2	5 0	2 0	3,0	348-523	673-1182	Faint rutting and showing all lanes
		3	5 5	2 0	3 0	534-670	817-1104	Well defined rutting 15M and 37M lanes with faint upheaval 25M lane Faint rutting 60M lane

[a]Designed for 550 Marshall Stability - From traffic results 4 8% - 5 4% is considered optimum binder content range
[b]Designed for 550 Marshall Stability - From traffic results 5 0% - 5 5% is considered optimum binder content range

from these data wearing surfaces of widely varying Marshall stabilities exhibited virtually the same service behaviors. In fact, in many cases the low Marshall stability mixes out-performed the high stability mixes. Certainly the minimum requirement of 500 lbs. Marshall stability cannot be justified on the basis of these results. This lack of correlation is believed by the writer to be due entirely to the fact that the Marshall test does not measure adequately the frictional resistance of bituminous mixes toward slow-moving or static compressive stresses.

Conclusions - The traffic tests conducted by the Waterways Ecperiment Station provide a wealth of information which merits detailed review by all highway engineers. Additional laboratory work should be undertaken wherein other stability tests are correlated against the results obtained in accelerated traffic tests. As the Marshall test is influenced predominately by the tensile or cohesive properties of bituminous mixtures under dynamic loading conditions, the results reported to date represent mainly a study of the effects of tensile properties upon mix performance.

At asphaltic binder contents considered optimum in the Vicksburg studies appreciable shearing distress took place in most of the surfacing mixes. These data and a few tests made on some of the mixes by

TABLE D

SUMMARY OF VICKSBURG RESULTS WITH ASPHALTIC CONCRETE MIXES OF VARIOUS MARSHALL STABILITIES ON HIGH SUPPORTING POWER FOUNDATION

Section	Unit	Item	Asphalt Content Wearing Surface %	Marshall Stability As Placed	Marshall Stability At 500 ± Coverages	Service Behavior Noted
					Mix 10[a]	
1A	1	1	5 4	172-210	353-449	Faint tire printing
		2	6 0	203-343	462-647	Faint rutting and shoving 37M and 60M lanes
		3	6 6	361-495	738-796	Faint rutting and shoving, faint upheaval in 37M lane
					Mix 11[b]	
1A	2	1	4 8	349-447	809-887	Faint rutting and shoving 37M lane
		2	5 4	375-738	846-1019	Faint rutting and shoving 37M and 60M lanes
		3	6 0	407-603	618-1111	Faint rutting 60M lane, well-defined tire printing 15M and 60M lanes
					Mix 12[c]	
1A	3	1	4 8	426-590	912-1250	Well-defined tire printing 15M lane, faint printing 60M lane
		2	5 4	774-1006	1062-1387	Faint rutting and shoving 37M and 60M lanes, well-defined printing 60M lane
		3	6 0	448-740	737-966	Faint rutting and shoving 15M and 60M lanes, well-defined printing all lanes
					Mix 13[d]	
2A	1	1	6.0	216-259	495-570	Faint rutting and shoving 37M and 60M lanes, well-defined printing 15M lane
		2	6 8	369-415	339-707	Approximately same as above
		3	7 5	337-400	357-625	Approximately same as above
					Mix 14[e]	
2A	2	1	5 7	276-314	719-801	Faint rutting and shoving 60M lane, tire printing other lanes
		2	6 4	398-463	825-859	Faint rutting and shoving all lanes
		3	7 1	357-414	473-514	Well-defined rutting 15M lane, faint shoving with well-defined printing other lanes
					Mix 15[f]	
2A	3	1	4 5	319-427	663-973	Faint rutting and shoving all lanes
		2	5 0	727-736	1043-1148	Well-defined rutting 15M lane, faint rutting and shoving 37M lane
		3	5 5	466-535	866-1173	Faint rutting and shoving 37M and 60M lanes

[a]Mix 10 Crushed limestone - Designed for 150 Marshall Stability - Optimum Binder Range reported as 6 3-7 0
[b]Mix 11 Crushed limestone - Designed for 350 Marshall Stability - Optimum Binder Range reported as 5.4-6.0
[c]Mix 12 Crushed limestone - Designed for 550 Marshall Stability - Optimum Binder Range reported as 4 8-5.4.
[d]Mix 13 Uncrushed gravel - Designed for 150 Marshall Stability - Optimum Binder Range reported as 6 0-6.8
[e]Mix 14 Uncrushed gravel - Designed for 350 Marshall Stability - Optimum Binder Range reported as 5 7-6 4
[f]Mix 15 Uncrushed gravel - Designed for 550 Marshall Stability - Optimum Binder Range reported as 5 0-5 5

other test methods indicate that at "optimum binder content" most of the Vicksburg mixes possessed stability qualities below the standards required in many areas.

The Marshall test possesses the simplicity and portability desired for use during military operations. As it measures predominantly the tensile characteristics of a bituminous mix, it should be complemented with another test which measures the other important stress resisting factor, frictional resistance. The need for testing eight specimens to determine Marshall stability indicates that a more reproducible and precise test would be desirable. This lack of reproducibility is believed to result primarily because the test is made at an extremely high rate of strain. That is, small differences from specimen to specimen in the viscosity of the bituminous binder result in relatively large changes in the measured stability due to the nature of viscous resistance of asphalts.

The design criteria suggested by the Army Engineers do not appear to be justified on the basis of traffic test results. Likewise these criteria would eliminate as unsatisfactory many types of asphaltic paving mixes which have proven themselves suitable through many years of service.

J. T. PAULS[1]. - A tremendous amount of work has been done in this investigation. The data presented indicates the usefulness of the Marshall stability test as a tool in the design of bituminous mixtures. This test, unlike other stability tests in current use, provides direct information on the plastic properties of such mixtures. The same type of information, of course, is obtainable from the simple compression test when load-deformation curves are plotted.

The principal consideration leading to the selection of this particular test method was the portability of the test equipment required and its adaptability in conjunction with bearing test apparatus which has been adopted by the Army for field testing by advance engineer units. The weight of this consideration in connection with this special military problem is no doubt important. In most civilian highway work, however, design of bituminous mixtures is done in a central laboratory considerably in advance of the construction. Once the job formula has been established, frequent checkups by the plant inspector on aggregate gradation and asphalt proportioning are believed to be the most effective means of insuring uniform agreement with the design.

The Marshall test, like the compression test, the Hubbard-Field test, and the Hveem stability test is an empirical test, and like the others mentioned it measures cohesion and to some extent internal friction. Although these tests do not permit separate evaluation of these two forces, the test results reflect the influences of both forces in varying degrees. The influence of friction on the test values obtained by either the Hubbard-Field test or the Hveem stability test is probably greater than those obtained by the compression test or the Marshall test. In the two latter tests the principal force acting to resist displacement is cohesion.

[1]Principal Highway Engineer, Division of Physical Research, Public Roads Administration.

The importance of stability is recognized. Stability test provides valuable information on such factors as grading, bitumen content, and density. At the same time stability should not be considered the sole element of design. There are many other important factors that effect the durability and surface characteristics of bituminous highway pavements not brought out by determining the stability alone. For example the resistance of the pavement to the action of moisture might be of vital importance. Improvement of this resistance might necessitate the use of a particularly dense mixture, the use of aggregate of small particle size resulting in reduced pore size in the compacted pavement, the selection of a particular type of filler, or the elimination from consideration of certain aggregates because of their hydrophilic properties.

The hardness and toughness of the coarse aggregate is another important factor affecting the behavior of a bituminous pavement that is not brought out by a test for stability. Aggregate degradation under rolling and later under traffic may alter the particle size and gradation to such an extent that failure of the pavement due to revelling, bleeding, or instability results.

The durability of an asphaltic pavement is also greatly affected by the ability of the binder to resist alteration of its physical and chemical characteristics caused by exposure to air and sunlight. Depending on source and refining processes, asphalts differ considerably in this respect. Stability tests made on a mixture at the time of construction fail to provide reliable indication of the service behavior of some of the bituminous binders, due to their tendency to harden rapidly. When such materials must be used, bitumen contents somewhat higher than are indicated by stability tests may be selected, with a view to retarding the effects of the aging process by providing greater film thickness.

Much work has been done recently to develop a test to differentiate asphalts with respect to their resistance to

alteration with age. The thin film oven test and the Hveem weathering test are noteworthy results of this work and are recognized as having considerable value.

In conclusion, necessarily hasty examination of the data lead to the following observations.

1. The apparent sensitiviey of the Marshall stability test to such variables as bitumen content, density, type of aggregate, grading, etc. demonstrates its usefulness as a tool for designing bituminous mixtures with respect to the property of stability.
2. Substantially all of these papers emphasize the property of stability, to the exclusion of other equally essential properties.
3. The tentative design limits suggested in the papers were based on correlation with accelerated test tracks. The essential difference between the effects induced by the intensive test conditions and those obtained in normal service should be borne in mind in evaluating these limits.

HARRY M. REX[1]. - The greater part of this symposium is devoted to the development of design procedure for bituminous wearing course mixtures based on the use of the Marshall stability test, and to the correlation of tentative design limits with results obtained on test sections subjected to accelerated traffic effects. The value of any laboratory method for testing bituminous mixtures rests on

testing bituminous mixtures rests on

its ability to predict the service behavior of such mixtures, and in the present instance the decision to use accelerated field tests to furnish data that would allow selection of laboratory test values in the design procedure in the shortest time possible is wholly understandable.

A consideration tending to qualify somewhat the findings of investigations of bituminous mixtures based on accelerated traffic tests is, of course, the time element. The binder in a bituminous wearing course is in its most plastic condition at the time of construction. Thereafter, throughout its service life, aging of the pavement is accompanied by gradual alteration of the bitumen, evidenced by decreasing penetration and ductility. As the viscosity of the binder increases, the resistance to consolidation under traffic increases. It is conceivable, then, that the degree of consolidation noted for some of the mixtures in the test sections might never obtain in actual service, where repeated loadings might be spread out over relatively long periods of time, during which the bituminous cement would be becoming less plastic and the wearing course becoming progressively more resistant to compaction. Many of the mixtures that were designated as "plastic" or "border plastic" under the test conditions might therefore, under actual service conditions, prove to be entirely satisfactory and in some cases superior to those rated as "satisfactory."

Likewise, this consideration should, and doubtless will, be taken into account in making final selection of certain details in the laboratory compaction procedure. Density correlation between laboratory specimens and samples taken periodically from actual service installations will probably be found to be more significant than correlation based on samples from accelerated test tracks. While it may be held that design based on the latter type of correlation will be conservative, at the same time there is the possibility that such procedure may eliminate mixtures having bitumen contents in the higher ranges that would prove to be satisfactory in actual service. It is assumed that the studies will be extended to include correlation of the design limits suggested in the symposium with the service behavior of additional airfield wearing courses.

Good correlation between the density obtained by field compaction and the density obtained in the laboratory compacting procedure is important in the satisfactory use, for design purposes, of any type of stability test. In the Marshall test,

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this correlation would appear to be especially important, since the selection of bitumen content is based upon the stability, unit weight of total mix and aggregate only, percent of voids aggregate only and total mix, and percent voids filled with asphalt.

It is believed that, whenever possible, the laboratory work in studies intended to correlate laboratory and field densities should be started at the time of constructing the field jobs, at which time samples of the fresh uncompacted mixture can be used for laboratory compaction. Densities of samples taken in the field from the finished work may then be compared to those of original laboratory-compacted specimens, thus avoiding the necessity to reheat and recompact.

One of the authors makes an observation with respect to specifications governing the compaction to be attained by rolling during construction that is particularly worthy of emphasis. He points out that many engineers have used specifications requiring compaction that will result in pavement densities equivalent to from "90 to 95 percent of the theoretical density without realizing that they were neither specifying a definite amount of compaction nor obtaining sufficient compaction in many cases."

Expanding on this point, it should be noted that similarly expressed requirements may sometimes result in over-compaction, accompanied by excessive crushing of the aggregate. If the theoretical density of a voidless mass is used as the reference specific gravity in such requirements, care must be exercised in selecting the numerical relative density to be required on individual jobs, giving consideration to the volume requirements of the bitumen content selected, and the void content of the mineral aggregate when compacted to its densest condition. Use of the observed specific gravity of laboratory-compacted specimens of the mixture that is to be used on a particular job, as the reference density with which the density of construction samples will be compared, is a much more realistic and logical practice. Whether laboratory compaction is accomplished using the procedure described in the symposium or by other laboratory procedures is a matter of choice, provided that the results may be correlated with field compaction.

RAYMOND C. HERNER, *Civil Aeronautics Administration* - The vast amount of laboratory and field work so ably summarized in this symposium has given us a valuable tool for use in the design of bituminous mixtures and flexible pavements. It is a tool, however, which must be used with a proper regard for its limitations.

First, we must recognize the fact that the approach is strictly empirical and that the resulting conclusions can be considered valid only within the limits of the test conditions. The Marshall test does not measure any fundamental quality of the asphaltic mixture or pavement, but indicates a composite result of several qualities when a specimen is tested under certain arbitrary test conditions. It is entirely possible that these qualities are not given the same comparative weights in the test results as they are under service conditions.

In this connection we should note that the word "stability", when applied to the Marshall test, is somewhat of a misnomer. This is brought out clearly in the symposium, from which the following quotations are made:

> "Referring to both the stability and flow curves, it can be seen that values of equal stability can be selected both below and above the optimum asphalt content. The flow curve shows that such values do not represent equally stable mixtures." (Paper No. 1. Underlining added).
>
> "The stability value is not a satisfactory indication of the ability of a mix to resist displacement under traffic ---." (Paper No. 4. Underlining added).

This is shown also in the field tests where we find instances of asphaltic concrete mixes with Marshall "stability" values greater than 500 and initial flow values less than 20 which still were rated as plastic.

As indicated in the symposium, accelerated traffic tests do not permit any evaluation of durability. Both experiments and experience have shown that a comparatively high asphalt content will contribute materially to achievement of this desirable quality. In the proposed method of mixture design the asphalt content is determined by averaging the results obtained from certain laboratory tests. It would appear more desirable to set the asphalt content at the maximum amount which would satisfy the limiting values set up for the various criteria.

For instance, in Paper No. 6 a design example was worked out by the averaging method which gave a value of 5.7 percent for the selected asphalt content. If this percentage is increased to 6.6 the mixture still will meet the limiting requirements for Marshall stability, flow, voids total mix, and voids filled with asphalt. The unit weight will be reduced only one pound per cubic foot. If the criteria are valid such a mix should be satisfactory for resistance to traffic, and it should be superior to the 5.7 percent mixture from the standpoint of durability.

W. H. CAMPEN, *Manager*, Omaha Testing Laboratories - One of the major conclusions reached in this symposium states that asphaltic content and density are the principal controlling factors in the development of stability. My own observations in the laboratory and field confirm this conclusion in a general way. However, the type of aggregate is most important also. I wish to elaborate on this phase because in my opinion the symposium does not cover it adequately.

On this point the symposium states that the type of coarse aggregate is not important in mixtures of sheet asphalt and coarse aggregate unless the percentage of coarse aggregate is 40 or more. I agree that the effect of coarse aggregate is not pronounced until the percentage exceeds about 40 regardless of whether it is round or angular and I also agree that when the percentage of coarse aggregate is high enough to produce interlocking the angular aggregates are more effective in developing stability than rounded ones. But, the effect of angular aggregates in bituminous mixtures is not confined to coarse aggregate sizes.

For instance, we find that we can more than double the Hubbard-Field stability in sheet asphalts by replacing about 40 percent of the natural sands with angular ones. Furthermore, stone-filled sheet asphalt mixtures in which the minus No. 10 portion contains about 40 percent angular material possess from 300 to 700 percent as much stability as do those in which the minus No. 10 material is all natural sand.

Furthermore, asphaltic concretes containing about 50 percent uncrushed coarse aggregate are three times as strong when about 40 percent angular aggregate is included in the minus No. 10 material as when natural sands only are used in the minus No. 10 portion. Even when the coarse aggregate consists of angular aggregate the stability increases from 50 to 100 percent when about 40 percent angular material is included in the minus No. 10 portion.

All these examples show the important part played by angular fine aggregate. They also show that coarse angular aggregates are more effective in producing stability when used in conjunction with strong mortars than with weak ones.

This brief analysis shows definitely that the role of angular aggregates in the development of stability is of utmost importance. A complete study will show that a wide range of stabilities can be produced in all types of mixtures by including angular aggregates which may be found in natural deposits or which may be produced by crushing durable rocks or gravels. A rather comprehensive study of the effects of both fine and coarse angular aggregates is included in a paper entitled: "A Study of the Role of Angular Aggregates in the Development of Stability in Bituminous Mixtures" by W. H. Campen and J. R. Smith, and published in the proceedings of the Association of Asphalt Paving Technologists, Vol. 17, 1948.

The symposium also attaches importance

to a flow test. It is used as a measure of plasticity. I certainly agree that we need a test to measure the wetness of bituminous mixtures because many of us are wondering if in designing for stability we are not sacrificing durability. However, the flow test as finally used in the symposium establishes upper values only. This is unfortunate because I believe it even more important to specify the lower limit than the upper one. My own experience in designing for stability leads me to conclude that the chances are much greater for producing dry, brittle mixtures than for producing wet, flexible ones.

That part which shows the effect of traffic on the density of bituminous mixtures is no doubt the most valuable contribution of the symposium. The laboratory compactive effort finally adopted for the preparation of test specimens is high enough to be equivalent to that of high wheel loads. For that reason the highest density which can be developed in the field will probably never be higher than that obtained in the laboratory and consequently the voids will not become lower than the minimum obtained in the laboratory.

I think it is good engineering to establish a compaction method which applies a specified amount of energy. All of us who make stability tests by other methods should take advantage of the data furnished by the U. S. Engineers and modify our compactive efforts accordingly.

NATIONAL RESEARCH COUNCIL

The National Research Council is a cooperative organization of the scientific men of America. Its members include, however, not only scientific and technical men but also business men interested in engineering and industry. It was established in 1916 by the National Academy of Sciences.

The Charter of the National Academy of Sciences, under a special act of the Congress approved by President Lincoln in 1863, provides that "the Academy shall, whenever called upon by any Department of the Government, investigate, examine, experiment, and report upon any subject of science or art."

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HIGHWAY RESEARCH BOARD

The Highway Research Board is organized under the auspices of the Division of Engineering and Industrial Research of the National Research Council. Its purpose is to provide a national clearing house for highway research activities and information. The membership consists of 36 technical, educational, industrial, and governmental organizations of national scope. Associates of the Board are firms, corporations, and individuals who are interested in highway research and who desire to further its work.

The Board provides a forum for the discussion and publication of the results obtained by individual research workers; organizes committees of experts to plan and suggest research work and to study and correlate the results; publishes and otherwise disseminates information; provides a research information service; and carries on fact finding investigations. With the cooperation of the highway departments of the states and Territories and the Public Roads Administration, the Highway Research Board conducts a Highway Research Correlation Service, to aid the many highway research agencies to correlate their work through personal visits, conferences, committee work, and dissemination of pertinent information.