210

HIGHWAY RESEARCH BOARD

Bulletin 210

Flexible Pavement Design--

Research and Development 1958

National Academy of Sciences—

National Research Council

publication 663

HIGHWAY RESEARCH BOARD

Officers and Members of the Executive Committee

1959

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2101 Constitution Avenue Washington 25, D. C.

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Flexible Pavement Design--

Research and Development 1958

PRESENTED AT THE
Thirty-Seventh Annual Meeting
January 6-10, 1958

1959

Washington, D. C.

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Contents

Notes on the Corps of Engineers' CBR Design Procedure

CHARLES R. FOSTER, Chief, Flexible Pavement Branch; and
R. G. AHLVIN, Chief, Special Projects Section; Flexible Pavement Branch, Soils Division, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi

This paper begins with the requirements of a design method and attempts to explain the Corps of Engineers' adoption and use of the CBR design procedures. Background history of the development of these procedures is included and an idea of the magnitude of the supporting research program is conveyed. Details as to the present capabilities of the CBR design method are given and alternate design procedures are briefly discussed. In an appendix the CBR procedures are summarized as to basic concept.

● THE CALIFORNIA BEARING RATIO (CBR) design procedures as practiced by the Corps of Engineers are the primary concern of this paper; however, it is believed desirable to point out that the Corps of Engineers' flexible pavement design procedure embodies two features which deal with the pavement structure and a third which deals with the bituminous mixture. These are as follows:

1. Each layer must be thick enough to distribute the stresses induced by traffic so that when they reach the underlying layer they will not overstress and produce shear deformation in the underlying layer. The CBR procedures are used for determining the thickness required to prevent shear deformation in the underlying layer. This paper is concerned primarily with this problem, which is termed "thickness design."
2. Each layer must be compacted adequately so that traffic does not produce an intolerable amount of added compaction. The modified AASHO laboratory compaction test and construction specifications requiring the proper percentage of laboratory density are used to design against consolidation under traffic.
3. The flexible pavement must have a wear- and weather-resistant medium as a surface that will not displace under traffic. The Corps of Engineers' design procedures using the Marshall stability test are used to design the bituminous paving mixtures to produce a wear- and weather-resistant surfacing that will not displace under traffic.

The basic concepts of a thickness design procedure adapted to the Corps' uses should be noted. These are:

1. The designs must be based on laboratory tests because extensive field test sections are not feasible, particularly in the theater of operations.
2. The designs must be based on procedures which simulate prototype conditions. Samples must be compacted to prototype density and adjusted to future moisture conditions before being subjected to tests. It should be noted that a thickness design procedure has two basic parts: (a) determining the protective thickness required for a soil with a given CBR value, and (b) estimating the CBR the soil will develop after it has been placed in the pavement system and the moisture content has become adjusted to the weakest condition.

CBR DESIGN PROCEDURES

The CBR design procedures have been described in detail elsewhere (1). Briefly, the procedures consist of determining the CBR of the material to be used in a given layer and the application of the CBR to design curves to determine the thickness required above the layer to prevent shear deformation in the given layer during prototype traffic.

The CBR test is conducted by forcing a 2-in. diameter piston into the soil. The load required to force the piston into the soil 0.1 in. (sometimes 0.2 in.) is expressed as a percentage of the standard value for the same penetration in crushed stone. The test can be performed on samples compacted in test molds, on undisturbed samples, or on material in place. The test should be made on material in a state which represents the prototype condition that will be most critical from a design standpoint.

The CBR design procedures as currently used by the Corps of Engineers and the thickness design curves for single-wheel loads, 100-psi tires, are given in the Appendix.

ORIGINAL ADOPTION

The adoption of the CBR method of thickness design for flexible pavements is discussed by McFadden and Pringle(1). They state that during the latter part of 1940 the responsibility for design and construction of military airfields was assigned to the Corps of Engineers. It soon became apparent that a uniform method of thickness design was needed. Several methods then in vogue were based on the bearing capacity of the subgrade as measured by plate-bearing tests. In order to evaluate the use of the plate-bearing test in determining subgrade bearing capacity, special field investigations were made at Langley Field (Va.), Bradley Field (Conn.), and on a Virginia highway test section. The results of these investigations indicated that:

1. The length of time required to develop a satisfactory plate-bearing test procedure would preclude its use in the war emergency program then being faced.
2. In the field plate-bearing test, the proper deflection to determine the "bearing capacity" depends on the basic assumptions in the formula and varies according to combinations of many factors.
3. In most cases, the results of the plate-bearing test would not be applicable to the soil-moisture conditions expected ultimately to develop below a pavement, and it would be difficult to develop a satisfactory method for adjusting the test results to the various moisture conditions.

Methods based on plate-bearing tests made on the surface of the pavement also were studied. Field tests led to the conclusion that the same factors which must be considered in using the test for subgrades must also be considered for tests on the pavement. In addition, the compressibility of the base and pavement entered the problem, and it is practically impossible to differentiate between the deflection due to shear deformation and that due to compression.

From these studies it was definitely concluded that adaptation of an empirical method that had been proved for highway loading was the best solution. This decision narrowed the field and, after some months of investigation of suggested methods, the principles used by the California Highway Department in designing flexible pavements for highway loading, known as the CBR method, were adopted tentatively. The controlling reasons for the adoption were many. Among these reasons were the following:

1. The CBR method had been correlated to the service behavior of flexible pavements and construction methods and successfully used by the State of California for a number of of years.
2. It could be more quickly adapted to airfield pavement design for immediate use than any other method.
3. It was thought to be as reasonable and as sound as any of the other methods investigated.
4. Two states were known to have methods of a similar nature that had been successful.
5. The CBR could be obtained with simple portable equipment either in the laboratory or in the field.
6. Testing could be done on samples of soil in the condition representative of the future moisture condition under most pavements.

DEVELOPMENT OF CBR METHOD FOR AIRFIELDS

In 1942, O. J. Porter, George Bertram, the late T. A. Middlebrooks, and Arthur Casagrande assembled in the Office of the Chief of Engineers, in Washington, D. C., at the request of engineers of the Airfields Branch, OCE, to extrapolate the existing CBR design curves for 7,000- and 12,000-lb wheel loads to wheel loads of 60,000 lb. These men drew upon their knowledge of soils and pavement requirements and prepared design curves which later experience has proved to be remarkably good.

Work began almost immediately on the verification of these extrapolations and on studies leading to the improvement of the CBR test procedures. Initially, the work was assigned by the Airfields Branch, OCE, to the various district offices of the Corps and to the Waterways Experiment Station. After the initial rush of work was out of the way, the Airfields Branch developed well-planned procedures for operating the investigational program which included written long-range and short-range programs, written plans of tests, annual review of programs by a board of consultants, and frequent review of the program with responsible engineers in the Air Force. The requirements for airfield pavements have been constantly increasing due to increased loads and tire pressures and the investigational program has been concerned with providing design criteria to meet these increased requirements. Engineers of the Airfields Branch, OCE, who have been closely associated with this work are Gayle McFadden, T. B. Pringle, R. M. Haines, and F. B. Hennion. F. L. Meara of the U. S. Air Force and S. J. Buchanan, consultant to the U. S. Air Force, have assisted with the reviews. The programs have included the following:

1. Accelerated traffic tests to verify the CBR design curves for single wheels.
2. Study of CBR test procedures.
3. Investigation of failed and satisfactory airfield pavements and comparison of results with design criteria.
4. Development of a more rational method of thickness design.
5. Extension of design procedures to multiple-wheel assemblies and high-pressure tires.
6. Effect of load repetitions.

A board of consultants was retained and assembled at regular intervals to review the investigational programs and to assist in analyzing the test data. The consultants have been outstanding individuals in the fields of soil mechanics, highways, and airfields. Those retained to review the investigational program as a whole included Authur Casagrande, R. E. Fadum, J. L. Land, T. A. Middlebrooks (deceased), O. J. Porter, R. B. Peck, P. C. Rutledge, D. W. Taylor (deceased), H. M. Westergaard (deceased), and K. B. Woods. Assisting in various phases of the program are R. A. Barron, G. E. Bertram, D. M. Burmister, M. J. Hvorslev, N. M. Newmark, Gerald Pickett, and R. R. Philippe. Some of the consultants are from within the Corps of Engineers' organization.

In 1943 the Office of the Chief of Engineers established the Flexible Pavement Laboratory at the Waterways Experiment Station. The investigational program on thickness design has been the responsibility of this laboratory, although some of the test sections have been constructed elsewhere. Engineers who have had major responsibilities in connection with the thickness design programs at the Flexible Pavement Laboratory have been W. J. Turnbull, W. H. Jervis, W. K. Boyd, J. B. Eustis, S. J. Johnson, W. G. Shockley, and C. R. Foster.

The studies that have been conducted for the development and improvements of the thickness design procedures and the major findings are summarized briefly.

Accelerated Traffic Tests

As noted previously, immediately following the extrapolation of the 7,000- and 12,000-lb design curves to curves for higher loadings, a series of accelerated traffic tests was initiated to validate the extrapolations. Special test sections were built at Stockton Field, Calif. (2, 3), Barksdale Field, La. (4), Eglin Field, Fla. (5), and Langley Field, Va. (6), and subjected to accelerated traffic with wheel loads up to

50,000 lb, which was the limit of the available equipment. Accelerated traffic tests also were conducted on existing pavements at eight airfields (7). In 1945 and 1946, a second test section was constructed at Stockton Field and subjected to traffic with wheel loads up to 200,000 lb (8). Tire pressures were generally 100 psi or less, except for the heaviest loads at Stockton No. 2. These studies permitted comparisons between the thickness design curves and the performance under traffic. It should be noted that the comparisons were based on the in-place CBR that existed during the traffic period. As mentioned previously, the problem of developing a design procedure is two-fold. One aspect is determining the thickness required over a soil with a given CBR; the other, is estimating the CBR that a soil will have in prototype conditions. The accelerated traffic tests provided information for only the first part of the program. The comparisons for wheel loads up to 50,000 lb are given by Foster in the CBR symposium (1). He shows that the results were in good agreement for loads below 30,000 lb, but the data indicated that additional thicknesses in the order of 4 to 5 in. were needed for the heavier loads. The results of the Stockton No. 2 data, together with results of behavior of pavements at actual airfields and theoretical studies, were used to make adjustments in the single-wheel load curves which included the increases for the heavier loads. These curves, shown in Figure 43 of the CBR symposium (1), were placed in the July 1951 issue of the "Engineering Manual for Military Construction" (9), and have been used since then with no significant changes. It is considered that the design curves for the single-wheel loads at tire pressures of 100 psi and less are adequately validated, and no accelerated traffic tests have been made for single-wheel loads and tire pressures of 100 psi since the Stockton No. 2 tests.

CBR Test Procedures

As mentioned earlier, one of the first investigations made after adoption of the CBR design method was a comprehensive study of the procedures for preparing samples for the laboratory test and for conducting the penetration test. This study was made at the Waterways Experiment Station and was reported in 1945 (10). One outstanding result of the study was the procedure for compacting samples at a range of compactive efforts and water contents and the plotting of the CBR results to show the variation of CBR with density for equal values of molding water content. These procedures permit a more realistic estimate of the CBR that will develop in the prototype than is possible with any other method. The procedures are described in more detail in the Appendix.

The testing procedures that were developed as a result of the Waterways Experiment Station study (10) were included in the 1946 issue of the Engineering Manual and have been used since that time with only minor modifications. The procedures work well for fine-grained soils, but the laboratory CBR obtained on gravelly soils tends to be higher than is developed in the prototype. The difference is due to the processing that is necessary when material occurs in excess of ¾-in. maximum size, and to the effect of the mold. Studies have been made of these problems, but no satisfactory test procedures have been developed. To produce satisfactory designs, the CBR test procedures for coarse-graded materials are being supplemented by gradation and Atterberg limits requirements for CBR design values above 20. These supplementary requirements are explained in the Appendix.

Surveillance Studies

Because the real proof of a design procedure is the performance of pavements designed by the procedure under actual traffic, the Flexible Pavement Laboratory has made numerous investigations of pavements at airfields. Failed pavements have been investigated to determine the reason for failure and to obtain a comparison with the behavior and the design criteria. Satisfactory pavements receiving heavy traffic have been investigated for a comparison of existing conditions with the design criteria. Foster (1) presents data collected through 1949 in this study. In general, the results show that the thickness criteria are satisfactory, and in many cases conservative. Field investigations of failed and satisfactory pavements have been continued throughout the years and show the same trend; more are planned for the future.

Rational Design

From the outset, the consultants were of the opinion that the design procedures for flexible pavements would have to be, initially at least, empirical in nature. Some of the consultants doubted that it would ever be possible to develop a truly rational design procedure because of the complexity of the problem, but all agreed that the problem should be studied. Pressure cells and deflection gages were installed in the early test sections and theoretical computations of pressures were made at the Waterways Experiment Station. After a study of these data, the Flexible Pavement Laboratory recommended in 1945 a stress-distribution study which had as its basic purpose the development of a more rational method of flexible pavement design. The requirements for a truly rational design have been established, as follows:

1. Compute the stresses (or strains) induced in a given layer.
2. Submit this layer to a test to measure its true ability to resist the stresses (or strains).
3. Compare stress (or strain) resisting ability with induced stress and express the comparison as a ratio or a factor of safety.

It should be noted that the rather severe problem of how to compact a soil to its future prototype density and adjust the water content to a future weakened condition must be considered in the rational method or in any other method.

Two carefully instrumented test sections (one a sand, the other a clayey silt) have been constructed at the Waterways Experiment Station and subjected to loads over a wide range of conditions. Soil conditions in each were homogeneous. The results (11, 12) show that measured stresses agree closely with those computed using theory of elasticity for low loadings, but show deviations from theory for high loadings. The deviations were greater for the sand than for the clayey silt. Instrumentation difficulties were suspected in the tests made in sand, and recent studies have been concerned primarily with the accuracy of the measurements.

Samples of the soil from the two test sections were subjected to laboratory triaxial tests, and the laboratory stress-strain curves were compared with field stress-strain curves. A significant feature of these tests is that the field stress-strain curves obtained in these studies are believed to be the first ever obtained. The laboratory stress-strain curves obtained from the standard triaxial test showed wide deviation from the field curve. Experimental procedures were tried, and reasonable agreement between the laboratory and field stress-strain curves was obtained by duplicating in the laboratory the relation of vertical to lateral stress that had been measured in the field. Because, however, relationship of vertical to lateral stress is not known, except for the two soils tested, this procedure has little application at present. Future plans include the construction and testing of layered systems. The results will be compared with values computed by Burmister's layered theory (13) for possible validation or modification of the theory.

In addition to the tests previously described, numerous computations of theoretical stresses and deflections were made. One item of particular importance developed from these studies was the finding that the CBR design curves for CBR values below about 20 had a pattern similar to that exhibited by the load, depth, and stress relationship of the theory of elasticity. Fergus (1) showed that the CBR design curves for a given tire pressure could be expressed as $k = \sqrt{\frac{z}{P}}$, where z is depth, P is total load, and k is a constant depending on the CBR value. This relationship has been studied further and improved over the years. It has been of inestimable help in the development of the thickness design curves, as it has permitted comparison of service behavior data for a wide range of wheel loads on one plot as shown in Figure 40 of the CBR symposium (1) and has aided in adjusting the curves to obtain the best possible agreement with the data. Relationships similar to these have been used in adjusting the design curves to other tire pressures and for multiple gear configurations as discussed subsequently.

In summary, the status of the rational design studies is as follows:

1. Induced stresses can be computed for homogeneous sytems with a fair degree of accuracy; the degree of accuracy would probably be satisfactory for cohesive materials, but not for cohesionless materials.

2. Triaxial tests as usually run do not measure true stress-strain relationships. Triaxial tests can be run which duplicate field stress-strain conditions for the cases where the stress-strain conditions are known. Universal procedures for the tests have not been developed.

Although the stress distribution studies have not as yet produced a rational design method, they have produced the means for translating the single-wheel design curves into curves for other conditions of loadings. The studies have also permitted evaluation of the several so-called "rational" design procedures that have been proposed in the technical literature.

Multiple Wheels and High-Pressure Tires

The growth of aircraft during the latter part of World War II and subsequently has led to the development of multiple-wheel assemblies and high-pressure tires by the aircraft designers. A test section to compare the effects of the dual-wheel assembly used on the B 29 aircraft with a single-wheel assembly of equal load was constructed at Marietta, Ga. (14). The results were used to develop design curves for the B 29 aircraft. Subsequent study produced theoretical procedures for resolving the single-wheel curves into curves for multiple-wheel assemblies. These procedures are described by Foster and Boyd in the CBR symposium (1). The Stockton No. 2 test section also included limited study of multiple-wheel assemblies. In 1949 and 1950, a test section was constructed at the Waterways Experiment Station to verify the theoretical resolutions of single-wheel curves into multiple-wheel curves. These tests (15) indicated that the theoretical procedures gave thicknesses which were slightly unconservative. A complete reanalysis (16) of all data resulted in a more rational method of developing multiple-wheel design curves by adjusting the thickness for a given multiple-wheel load on a given subgrade to produce a deflection in the subgrade equal to that produced by the load when carried on a single wheel. The curves produced in this manner are in good agreement with the traffic test section data. It should be noted that this theoretical treatment stemmed from the rational design studies.

A similar procedure was used to adjust thickness requirements for high-pressure tires. Test sections (17) were constructed in 1949-1951 at the Waterways Experiment Station to check these procedures. The tests included tire pressures in the range of 200 psi and, as a result of these studies, the design curves are considered adequate for tire pressures up to 200 psi.

Effect of Repetitions

In the theater of operations, the thickness must be limited to that barely necessary to support the operation. Therefore, design criteria were needed for limited usage. A study of available data developed a relationship between volume of traffic and the percentage of full design thickness necessary to support the traffic. This concept is presented in Figure 41 of the closure paper to the CBR symposium (1). Improvement and modification of the concept have permitted the development of thickness design criteria for the following operations:

Type	Nominal Duration	Nominal Coverages
Assault	1 day	6
Emergency	2 weeks	40
Minimum	6 months	700
Full	2 years	2,000
Capacity	More than 10 years	5,000
Channelized	More than 10 years	30,000

Field Moisture Studies

Pavements must be designed not merely for the subgrade strength existing at the time of construction, but for the worst conditions expected in the future. Therefore, some evaluation of these future conditions is necessary. The CBR procedures make use of tests on soaked samples to take care of this condition. In February 1945 the Flexible Pavement Laboratory undertook a field moisture study which was intended to develop a better understanding of moisture conditions under flexible pavements. Airfields in various climatic zones were visited repeatedly in various seasons and in successive years. Test pits were opened and the necessary samples taken to evaluate mosture, density, and CBR. In addition, moisture cells have been installed in some instances and closely spaced periodic readings made of the field moisture. Results reported through November 1952 (18, 19) show that the 4-day soaking test is conservative for nonplastic or slightly plastic materials, but is about correct or slightly conservative for plastic or very plastic materials.

Present Status of CBR Procedures

In summation, the investigational work accomplished to date has yielded the following results:

1. Thickness design curves. Design curves are available which can be adjusted theoretically to any condition of gear configuration, tire pressure, and repetition. The design curves are validated for a range of tire pressures up to 200 psi, tire loads to 200,000 lb, and repetitions up to 5,000 coverages.
2. Determining prototype strength. Sample preparation and test procedures are available by which materials can be compacted to prototype densities and adjusted to a future condition of water content. When used with fine-grained soils, the procedures give results which are satisfactory or slightly conservative for the plastic materials and are conservative for nonplastic and slightly plastic materials. The procedures have been supplemented by gradation and Atterberg limits tests for gravelly soils.

OTHER DESIGN PROCEDURES

The Flexible Pavement Laboratory has followed closely the technical literature on flexible pavement design and has studied those design procedures which have been used or proposed for use. The Highway Research Board Committee on Flexible Pavement Design has reported (20, 21) on the various design procedures used by the organizations in the United States. These procedures can be grouped into four general categories as follows:

1. Procedures which can use an index based on soil constants or soil classification, such as the Bureau of Public Roads' group index or the CAA rating.
2. Procedures which use a physical test to obtain an index of the strength, such as the North Dakota cone or Florida bearing test.
3. Procedures which use a form of shear test, such as direct shear or triaxial shear.
4. Procedures which use a plate-bearing test.

The first two groups are not discussed further herein.

Shear Tests

The California Division of Highways uses the Hveem Stabilometer method of design (22), which utilizes the strength obtained from a modified triaxial test and a nomograph to obtain thickness. The Kansas method (23) uses a conventional type of triaxial test to measure strength. The problems associated with using a shear test were mentioned briefly earlier under "Rational Design." The major obstacles are the determination of the shearing stress induced in the system by the wheel loading and the determination of the normal stress which will be available to develop strength to resist this shearing stress. In both the Hveem and Kansas methods these problems are bypassed by conducting the strength test at a standardized lateral pressure and comparing designs based on these strengths with service behavior records. This is essentially an empirical procedure. The other procedures employing strength tests use similar methods to overcome the problems mentioned. These empirical procedures are workable, but design methods using them are only as good as the correlations which have been developed to verify the procedures.

Plate-Bearing Tests

In addition to the studies of plate-bearing tests which were made preparatory to adopting the CBR method, extensive plate-bearing tests were conducted in connection with the Barksdale Field test section (4). Also, study has been made of the plate-bearing tests made by the Bureau of Public Roads in its test track at Hybla Valley (report not published), the plate-bearing tests made by McLeod (24), the British Load Classification Number (LCN) System (25), and the Navy design procedure (26). It should be noted that the published correlations between the load required to produce a given deflection and the actual traffic-carrying capacity of the pavements are limited.

It is considered that the plate-bearing test could be used satisfactorily under the following circumstances:

1. The plate-bearing test procedures would have to be modified to produce large deformations (½ to 1 in.) in order to eliminate the effect of consolidation and measure only resistance to shear deformation.
2. The large deformations would require much larger loading equipment than available. At Barksdale Field, a load of approximately 28,000 lb was required to produce 0. 5-in. deformation with a 30-in. diameter plate on a clay subgrade. A load of 50,000 lb on a CBR of 5 did not produce 0. 1-in. deformation on the base course.
3. Correlations would have to be developed between the load at given deflections and actual traffic-carrying capacity.

The plate-bearing procedures have not been adopted by the Corps of Engineers because the correlations required between load predicted by the plate-bearing tests and actual traffic-carrying capacity would be very expensive and no feasible means for adjusting the water content in a full-scale test section could be devised. Also, the Corps of Engineers' contract procedures are such that no assurance could be had that the subbases and bases used in the test sections would be the same as those used in the final construction.

SUMMARY

The Corps of Engineers, after a study of available methods, adopted the CBR procedures for design of flexible pavements. Throughout the years, investigations have been continued to adapt the procedures to the needs of airfields and to the ever-increasing loads and tire pressures of military aircraft. At present, the CBR method is considered superior to other empirical methods because of the extensive correlations which have been developed for the method and because the method has been adapted to include variations in load, gear configuration, tire pressure, repetitions, and climatic conditions.

The engineers in the Corps responsible for this work have recognized the desirability of a more rational method, and investigations have been conducted to develop a more rational method. The work has produced many worth-while by-products, but a truly rational design method is not possible at the present time. Some of the problems may never lend themselves to truly rational treatment.

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Appendix

Summary of CBR Procedures Currently Used by the Corps of Engineers

This summary presents the basic concept of the CBR design procedures currently used by the Corps of Engineers; for details, the reader is referred to the current issue of the "Engineering Manual for Military Construction" published by the Office of the Chief of Engineers.

Where new construction is involved, which is the usual case, representative samples of the soils are tested for CBR value in the laboratory. A test program is prescribed which requires compaction of samples in 6-in. diameter test molds at three compactive efforts, approximating modified AASHO compactive effort, standard AASHO compactive effort, and an intermediate effort. Samples are prepared at a range of water contents for each effort. After compaction, the samples are soaked in water for four days under a surcharge load equal to the weight of the overlying base and pavement. After soaking, the samples are tested for CBR.

Basically, the CBR test consists of forcing a 2-in. diameter piston into the soil at a constant rate of 0.05 in. per minute and measuring both the load and the penetration. The load required to produce 0.1-in penetration (sometimes 0.2-in.) is compared to the standard load required to produce the same penetration in crushed stone. The load in the test is expressed as a percentage of a standard load.

When tests are conducted on samples compacted at a range of compactive efforts and water contents, the results produce a family of curves as shown in Figure 1. This family of curves shows the three-way relationships of molding water content, density, and CBR. These curves are then studied in view of the actual water contents and densities that can be expected in construction. The CBR values that will result from the combinations of water content and density are determined from the test curves, and a design CBR is selected, usually near the lower end of the range in CBR values. The shaded area in Figure 1 illustrates the range of water contents and densities that might

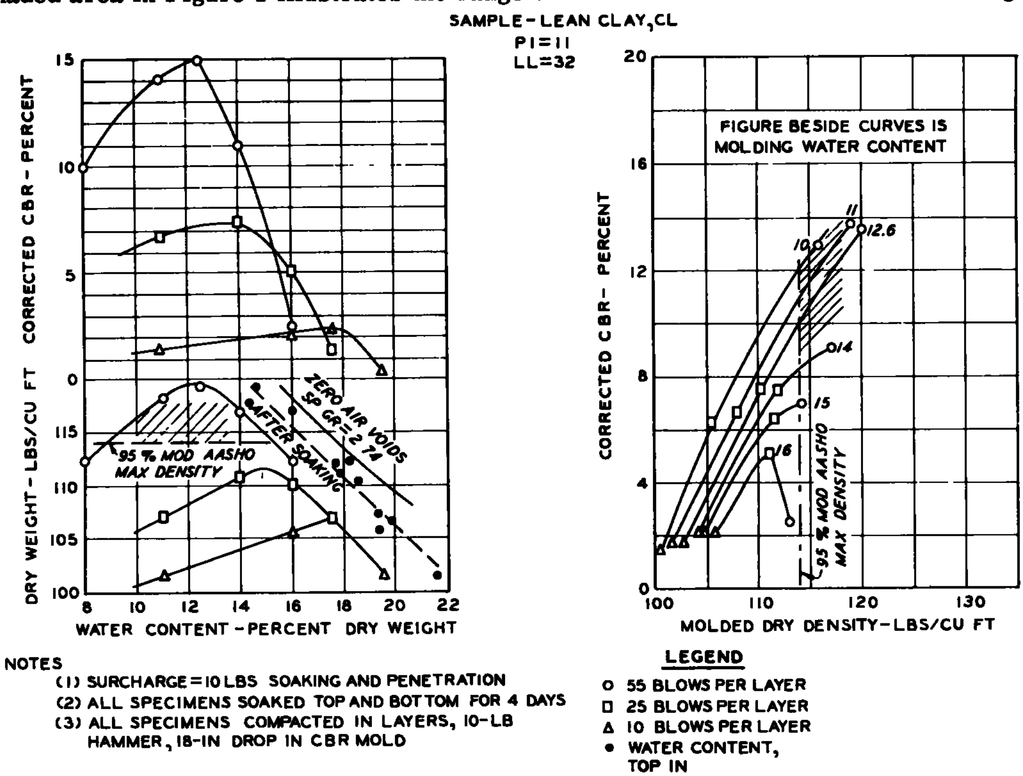

Figure 1. Recommended procedure for performing CBR tests for design.

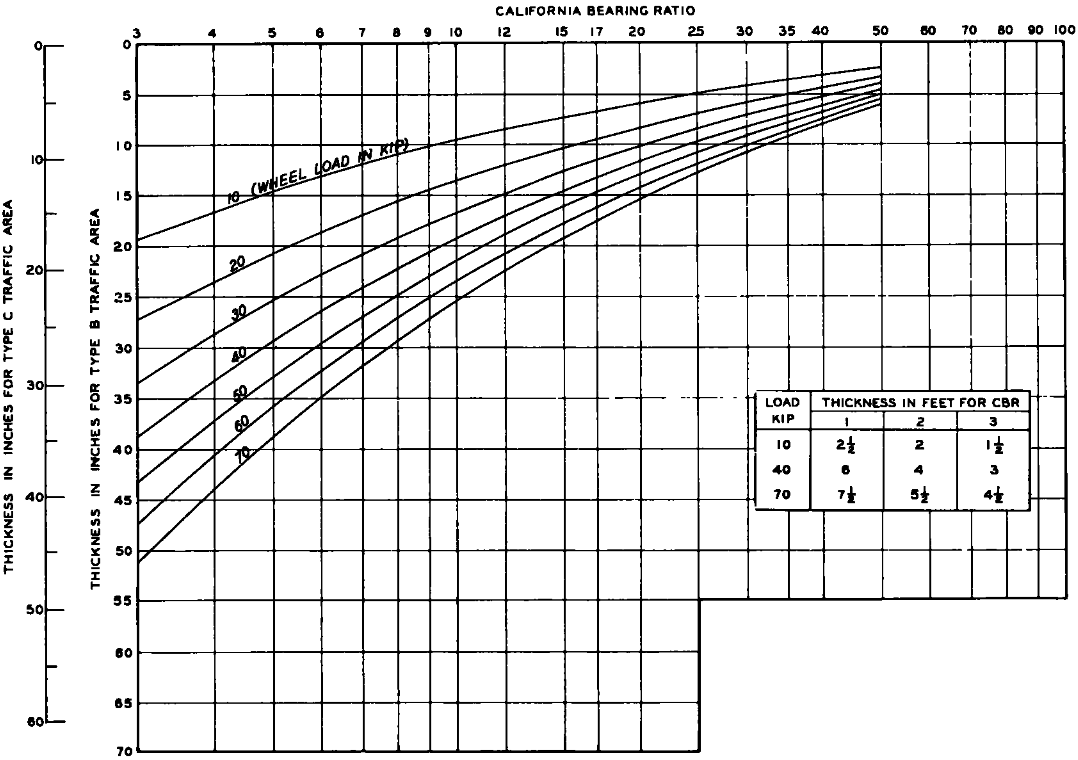

Figure 2. Flexible pavement design curves, single wheel, tire inflation 100 psi; types B and C traffic areas.

be expected for the soil tested. The CBR that will develop with this range of water contents and densities will vary from about 7 to 14. A value near the lower end of the range, say 8 or 9, should be used for design in this case.

Experience has shown that laboratory CBR tests on gravelly materials used for high-strength subgrades and subbase courses often show CBR values higher than are obtained in the prototype, primarily because of the confining effect of the mold. The CBR test has been supplemented by gradation and Atterberg limits requirements for gravelly materials as shown in the following table, which gives the gradation and limit values which must be met for the various design CBR values. In addition, the material must show a CBR in the laboratory test equal to or higher than the assigned design value.

	Maximum Permissible Value				
Design CBR	Max. Size (in.)	Gradation Requirements (% passing) No. 10	No. 200	Liquid Limit	Plasticity Index
40 - 50	3	50	15	25	5
30 - 39	3	80	15	25	5
20 - 29	3	100	15	25	5

The effects of processing base course materials and the effects of the 6-in. diameter mold in tests on base course materials are so great that laboratory CBR tests are not used for rating base course materials. Instead, arbitrary CBR ratings are assigned to materials meeting certain specification requirements. These ratings have been based on service behavior records and on in-place tests of materials that had been subjected to traffic. As an example, a graded crushed aggregate composed of both crushed-coarse and crushed-fine aggregate meeting relatively strict gradation requirements and having a plasticity index of 5 or less in place has been rated at a CBR of 100 percent. Stabilized-aggregate base, meeting essentially the same requirements except that the fine aggregate does not have to be a crushed material, has been rated at a CBR of 80 percent.

Figure 2 shows the CBR design curves for single-wheel loads and 100-psi tire pressures. The CBR of the soil being considered is applied to Figure 2 to determine the combined thickness of base and pavement. Curves are available for 200-psi tire pressures and for various multiple-wheel assemblies. Two thickness scales are shown in Figure 2; one for type B and one for type C traffic areas. The interior portion of the runway is designated as type C traffic area; the remainder of the airfield is designated as type B.

Discussion

W. H. CAMPEN, Manager, Omaha Testing Laboratories, Omaha, Nebraska — One of the principal points in the author's notes on the use of the CBR test results for purposes of thickness design deals with the elimination of the test for certain mixtures. The reason given for eliminating the test lies in the fact that laboratory results cannot be duplicated in the field. From actual field experience, it is agreed that field results are lower than those in the laboratory. However, the elimination of the test to measure base or subbase quality does not seem warranted.

Mr. Foster and his co-workers will no doubt remember that when the CBR test and its use for the estimation of thickness was proposed in 1942, the writer was in favor of adopting the test for the purpose of measuring quality, but was also very much opposed to its use in determining thickness of superimposed layers. He is of the same opinion now. The CBR test should be made on all subbase and base courses to determine their relative strengths. The indicated strengths should then be considered in determining total thickness on a given subgrade for a given wheel load.

As for the lack of CBR duplication in the field, there is at least one good reason, other than lateral support, why the field test might be lower. In the laboratory the sample has a rigid support in the steel mold base. In the field, however, the support consists of everything below the surface; all the base, subbase, and subgrade. Any consolidation or displacement in the subbase and subgrade would automatically decrease the CBR value in the base and any consolidation or displacement in the subgrade would reduce the CBR of the subbase.

CLOSURE, Charles R. Foster and R. G. Ahlvin — Mr. Campen points out the the laboratory CBR test has been eliminated for certain mixtures. It is desired to emphasize that this has been done only for high-quality base course materials that meet relatively strict gradation requirements and that have plasticity indexes of 5 or less. These materials typically show very high laboratory CBR values and also field CBR values equal to or higher than the design values that are arbitrarily assigned to them by the new procedures. For this reason, it was felt that the laboratory CBR test was no longer needed on these high-type base course materials. For granular materials of lower quality, the CBR test has been supplemented by gradation and limit requirements.

The authors concur with Mr. Campen that the steel base of the mold affects the results, in addition to the lateral support offered by the wall of the mold.

A Summary of Load Transmission Tests on Flexible Paving and Base Courses

RAYMOND C. HERNER, Chief, Airport Division, CAA Technical Development Center, Indianapolis, Indiana

The load transmission apparatus provides a means of conducting load tests on full-scale pavement sections under laboratory conditions. A segmented loading platform supported by coil springs is substituted for the natural subgrade to insure constant test conditions over long periods, and to permit quick and accurate measurement of vertical stresses transmitted through the pavement. For a given applied load, the maximum stress on the subgrade provides a convenient means of comparing the efficiencies of various pavement sections or evaluating the effects of various design variables.

The relationship of subgrade stress to applied load should vary with pavement thickness, contact area, and respective strengths (or stiffnesses) of the pavement and subgrade. An analysis of 814 loading tests on 123 pavement sections verifies the essential correctness of theoretical studies and provides numerical values for use in design or for futher study. These values are given in chart form, and represent a generalization of all pertinent test data. They may be used directly in a method of pavement thickness design based on a limiting subgrade stress or deflection, or to extend empirical designs into areas not adequately covered by service experience.

The triaxial test was used to compare strengths of the various materials (gravel, clay-gravel, sand, limestone, slag, and asphaltic concrete) used in the pavement sections. Although further correlation studies are in progress, the method outlined in this and previous papers is considered adequate for design purposes. The validity of the test data for use in highway pavement design will be checked by correlation with results of traffic tests on the AASHO Test Road.

THE LOAD TRANSMISSION testing project consists fundamentally of a series of static loading tests on full-scale flexible pavement sections. The pavements are supported by a mechanical subgrade in lieu of the natural earth. This provides a constant and uniform degree of support, and also facilitates measurement of vertical stresses and deflections transmitted to the subgrade.

Such an arrangement permits orderly long-term studies of pavement behavior under load, with each major variable controlled independently in a planned schedule of operations. The basic testing program has been underway for approximately seven years. During this period the relationship of load to subgrade deflection has been measured for almost 1,800 load applications on approximately 250 pavement test sections.

Several progress reports have been published (1, 2, 3, 4, 5, 6, 7, 8, 9). The present paper summarizes and discusses the results from all of the single-wheel loadings on various pavement sections supported by either a weak, medium, or strong subgrade. Major variables ranged within the following limits:

Applied load	2.5 to 60 kips
Pavement thickness	3 to 24 in.
Tire inflation pressure	40 to 200 psi

Pavement strength index	22 to 246%
Subgrade modulus	82, 150, and 300 lb/cu in.

Some of the detailed test data used in this analysis have already been recorded in CAA Technical Development Reports (4, 8); the remainder will be published in the near future.

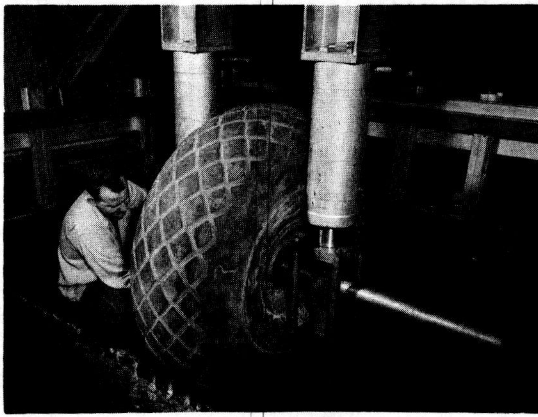

Figure 1. General view of load transmission apparatus.

APPARATUS, MATERIALS, AND PROCEDURE

The mechanical subgrade is about 10 ft square, with 3,600 segments supported individually by calibrated springs. The interior view (Fig. 1) shows a portion of the subgrade and a cutaway section of pavement, with an airplane tire in loading position. The subgrade pressure pattern is determined by reading spring deflections before and after loading the pavement. Each pavement section is subjected to a series of loads, increasing in magnitude, with the subgrade deflections measured after each load increment.

The tire inflation pressure is set at the desired value while the tire is entirely free of load, and is allowed to increase normally during the loading cycle. The actual inflation pressure at the heavier loads may be as much as 10 psi above the nominal pressure.

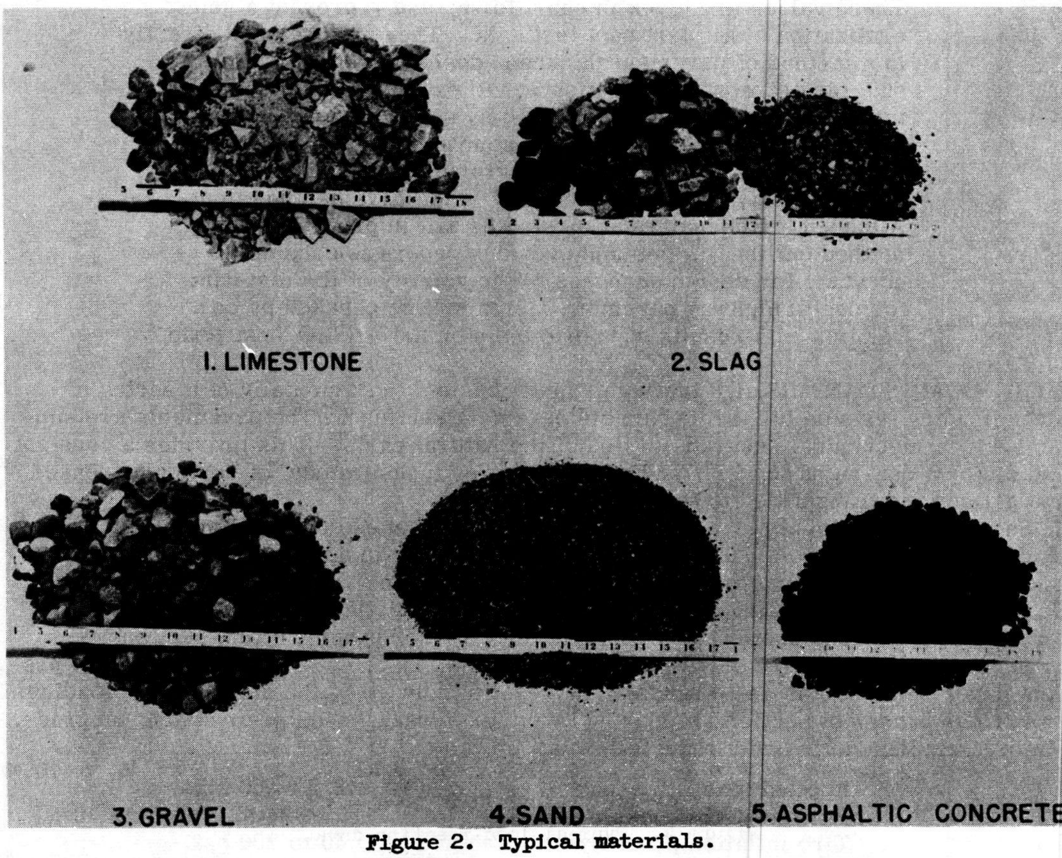

Figure 2. Typical materials.

A number of different airplane tires were used in the earlier tests. Although the size and geometric proportions of the tire appear to have some effect on the load-reaction curve, the effect is a minor one, at least for the range of tires used, and was not considered in the analysis.

Granular paving materials are blended and mixed in a pug mill mixer, placed in lifts of about 4-in. compacted thickness, and compacted by vibratory methods. Densities are comparable to those obtained in the field by rolling. A single material is usually used for the entire pavement depth, but some composite sections have been constructed.

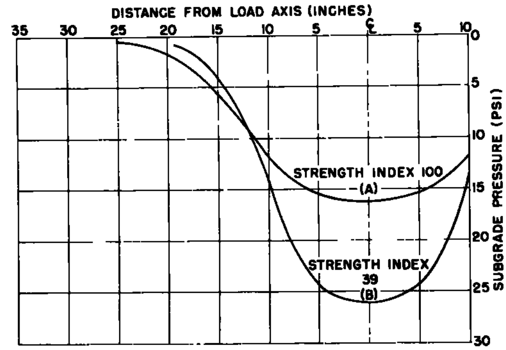

Figure 3. Cross-sectional views of pressure distribution.

The asphaltic hot-mix used to date has been obtained from commercial plants supplying material for highway and street work. Vibratory compaction of the asphaltic concrete has been supplemented by hand and pneumatic tamping.

Granular materials have included gravel, clay-gravel, sand, crushed limestone, and crushed slag. The materials were selected to provide wide ranges of grading, roughness, particle shape, and plasticity. They are not necessarily representative of

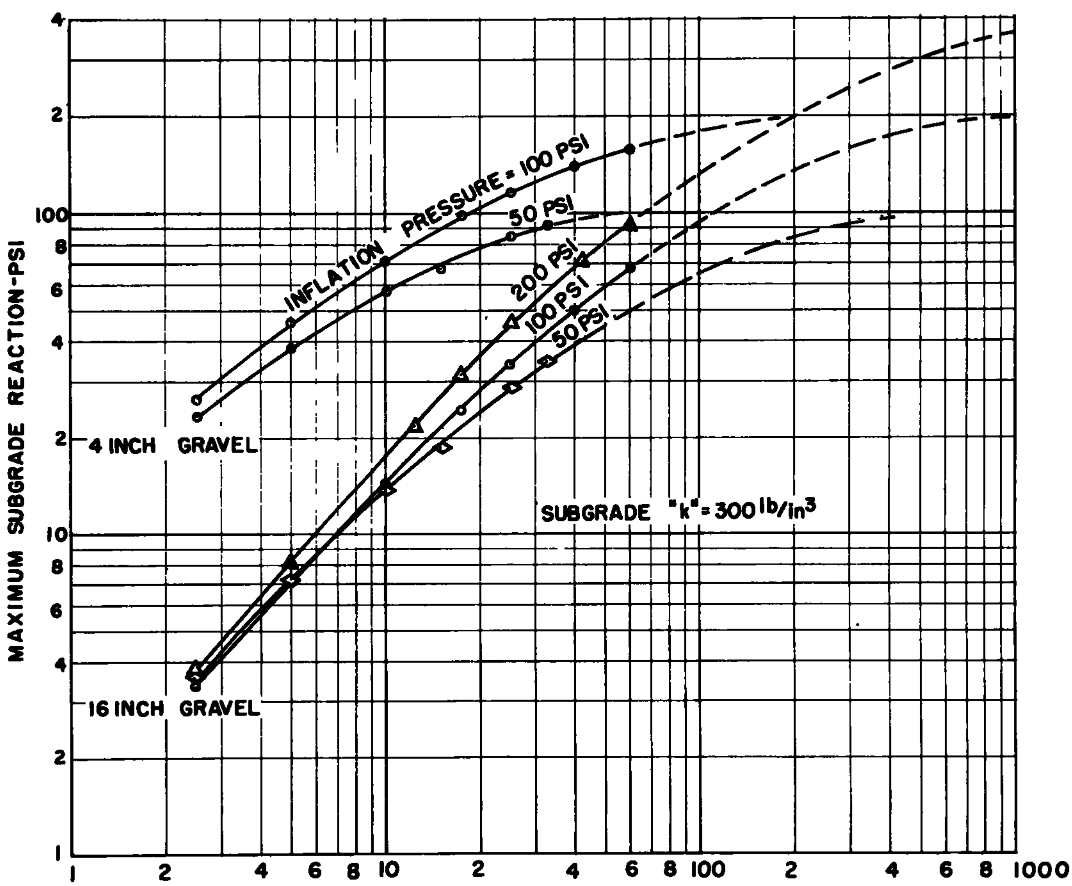

Figure 4. Typical load-reaction data.

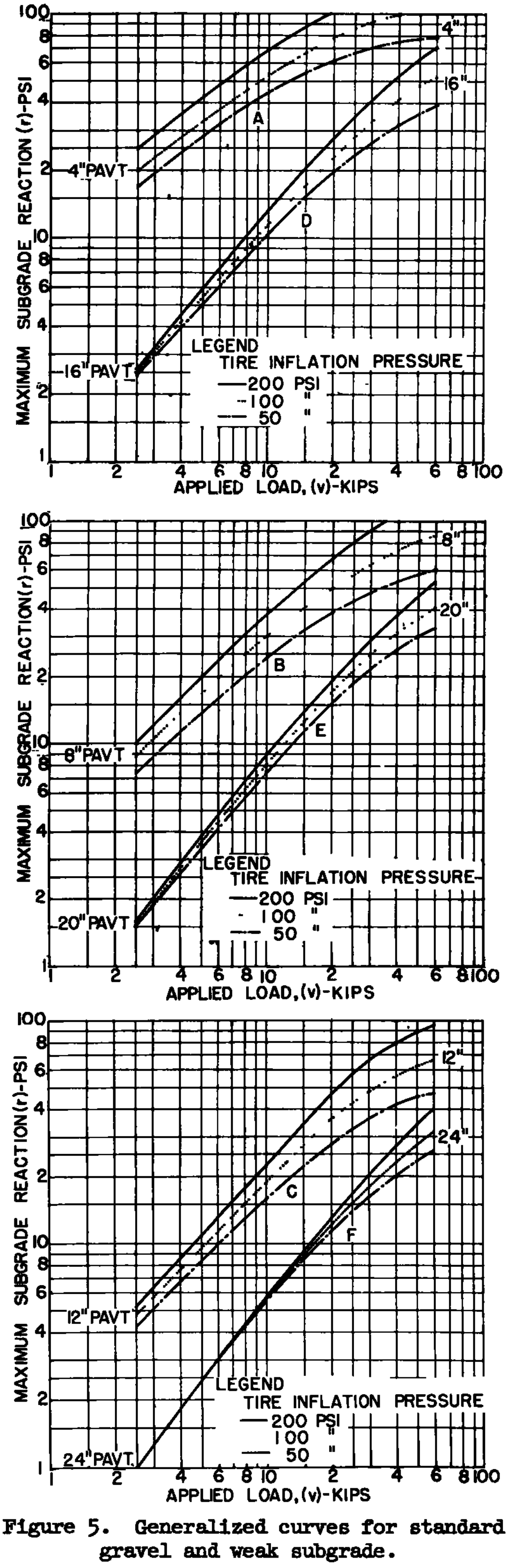

Figure 5. Generalized curves for standard gravel and weak subgrade.

the best or the average of the general types of material used. Typical samples are shown in Figure 2.

A portion of the paving mixture is set aside for construction of at least three triaxial specimens. These are designed to be of the same density, moisture content, and state of curing as the corresponding pavement section. When unplanned variations in these variables occur, the triaxial results are adjusted accordingly on the basis of judgment and experience.

There is some evidence that the lateral pressure for the triaxial test should vary, depending on the degree of confinement which the material will have in the pavement layer where used. The relationship and procedure now in use are defined later in the paper.

Inasmuch as the load transmission test itself is a static test, it follows that the triaxial tests should be run at a loading rate low enough to eliminate any dynamic effect on the indicated strength of material. This would involve loading at a strain rate of about 0. 01 percent per minute, and would require several hours to run one test. In order to achieve a reasonable production rate the granular materials and mixtures, all of which are affected to a similar degree by rate of loading, have been tested at a strain rate of 0. 5 percent. The asphaltic concrete specimens, much more sensitive to this effect, were loaded at a strain rate of 0. 02 percent, which gave comparative results. The indicated strengths under triaxial test are not absolute values but are used only in comparing each material with the standard.

DISCUSSION OF TEST DATA

The pressure pattern on the subgrade may be expressed conveniently in the form of pressure "contours." When loading with single tires, however, with the maximum value always occurring under the center of load, the pressure distribution may be shown more simply by use of cross-sectional views on the principal axes of the pattern. This has been done in Figure 3.

Although the entire distribution pattern is required for some purposes, such as studies of multiple loading, it often is possible to use the maximum value of the subgrade reaction as a convenient measure of comparative pavement performance. For example, pavement A of Figure 3 has done

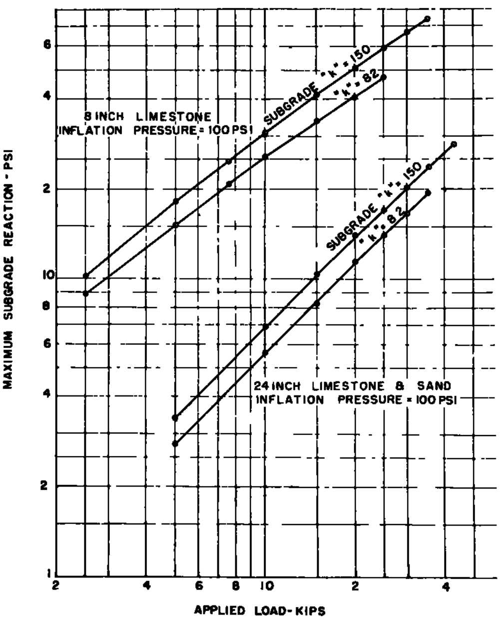

Figure 6. Effect of subgrade modulus.

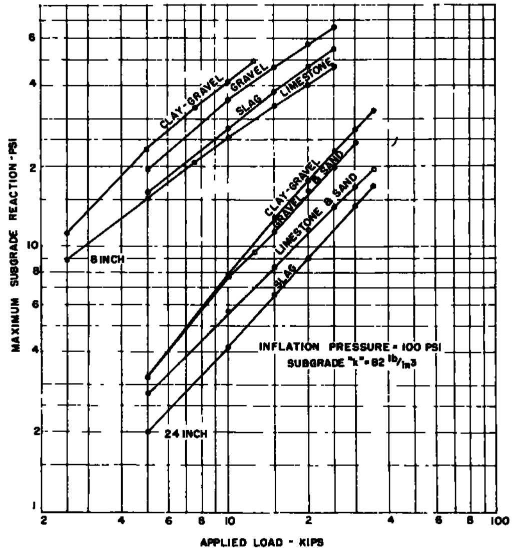

Figure 7. Effect of pavement strength.

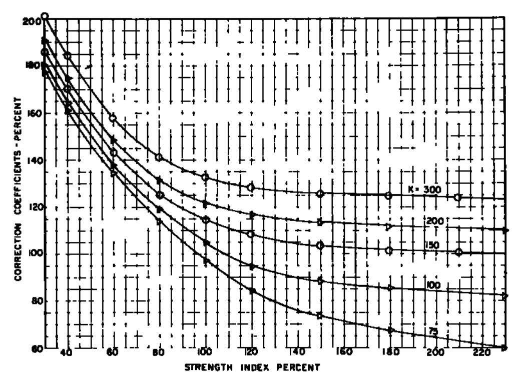

Figure 8. Correction coefficients.

a better job than pavement B in distributing load over the subgrade and this fact may be expressed by a numerical comparison of maximum subgrade reactions. This is the criterion used to measure pavement effectiveness in this report.

Figure 4 presents typical load-reaction curves plotted from actual test data. When plotted on log-log scales the bottom portions of the curves are nearly straight. The spacing and slope of the curves vary widely with pavement thickness. As the loads are increased the curves tend to flatten, reflecting the effect of increased contact area. If contact pressures on the pavement surface were uniform, and were equal to the initial tire inflation pressure, the maximum subgrade reaction would tend to approach this value as a limit when the applied load and corresponding contact area became infinitely large.

Lawton (9) showed that the contact pressure under a tire is far from uniform and may reach a maximum considerably above the nominal inflation pressure, even under moderate loads. The dotted portions of the curves in Figure 4 indicate that extrapolation of the curves to a value of twice nominal inflation pressure would not be unreasonable.

After compilation and analysis of all available data it was possible to construct the generalized curves of Figure 5 which by simple interpolation will give the relationship of maximum subgrade reaction to applied load for any combination of pavement thickness and inflation pressure within the limits indicated. These curves apply directly to only the standard gravel base course material and weak subgrade (k = 82). They are similar to those presented in CAA Technical Development Report No. 282 (8), but have been revised slightly for better over-all agreement with all the test data now available.

In order to generalize the curves for all conditions it is necessary to provide corrections for the effects of subgrade stiffness and the strength (or stiffness) of the paving material. Changes in either of these variables tend to move the loading curves parallel to themselves as shown in Figures 6 and 7. On a log-log graph this means that ordinates from one curve are related to those of another curve by a constant ratio. Values of maximum subgrade reaction taken from Figure 5 can be corrected for other subgrade and pavement strengths by simply applying correction coefficients taken from Figure 8.

Subgrade stiffness is indicated by use of the familiar modulus of subgrade reaction (k) used in Westergaard's theoretical analysis of rigid pavements. Use of this "heavy liquid" concept of subgrade support is particularly appropriate to the mechanical subgrade used in the load transmission tests.

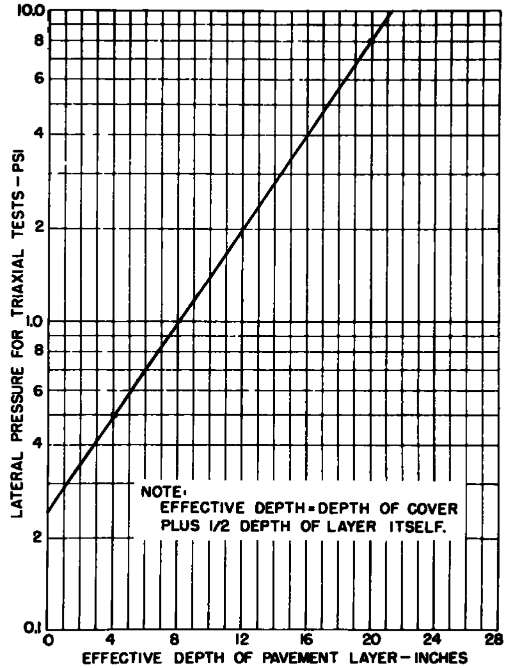

Figure 9. Determination of lateral pressure for triaxial tests.

The strength of the pavement section is expressed as a "strength index." This is the percentage of the strength of the gravel base course material used as a standard in preparing the curves in Figure 5. The strength index of a pavement section may be determined from triaxial tests in the following manner:

1. Divide the proposed pavement section into layers if necessary, each not more than eight inches thick, and each composed of only one type and quality of material.
2. Perform triaxial tests on specimens representing each layer of material, using a lateral pressure which is determined by the average depth of the layer in the pavement section. See Figure 9 for the lateral pressure to be used.
3. Divide the vertical pressure at failure, determined in Step 2, by the corresponding value for the standard gravel at the same lateral pressure. Values for the standard gravel are given in Figure 10, which also shows average curves for some of the other materials which have been tested.
4. If the pavement consists of more than one layer, take a weighted average of the ratios obtained for the various layers in Step 3. This is the strength index for the entire pavement section.

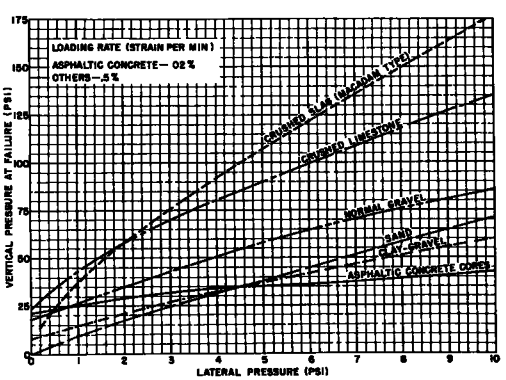

Figure 10. Triaxial test data--relationship of principal stresses.

It may appear that a modulus of deformation representing stiffness rather than ultimate strength should be a better criterion for comparing paving materials. This has not proved true in the analysis thus far, but is being studied further.

Values of subgrade reaction computed from triaxial test data by use of Figures 5 and 8 have been compared with the corresponding values actually measured in the load transmission tests. Deviations of measured values from computed values were plotted on probability paper and are in Figure 11. The plotted points approach a curve of normal distribution, indicated by a straight line on the graph. The standard deviation is 11 percent.

Of the 804 readings involved in the comparison, 93 percent fall within 20 percent of the computed values. Deviations greater than 20 percent occurred mostly in small readings where differences of one pound per square inch or less appear large when expressed percentagewise. Any closer correlation between computed and measured values apparently can be achieved only by use of a more complicated process, and does not appear justified from a practical point of view.

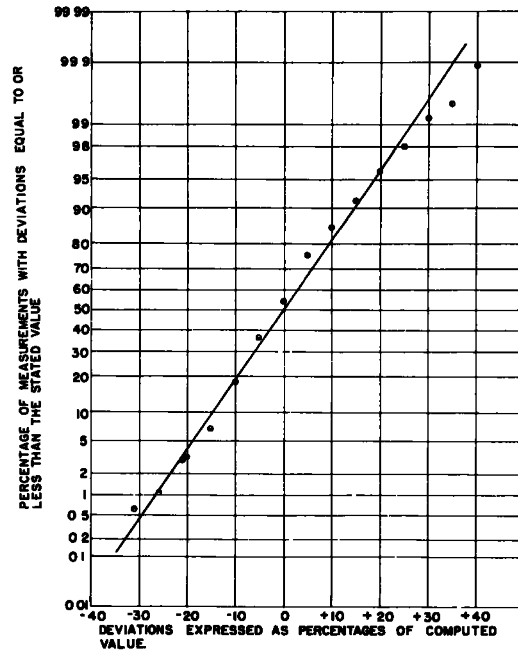

Figure 11. Deviation of measured vs computed values.

APPLICATION TO DESIGN PROBLEMS

From a qualitative standpoint the load transmission data add nothing new in the field of flexible paving design. Stated in general terms, the effects of pavement thickness, tire inflation pressure, pavement stiffness, and subgrade stiffness have been known for a long time, either through experience or through theoretical studies. What the load transmission project has done is to express these general relationships in specific figures which can be used in the design and evaluation of pavements.

It must be recognized, of course, that a pavement may fail from any one of three major causes, as follows:

1. Deterioration from weather or other environmental conditions.
2. Shearing failure or plastic flow within the pavement structure itself.
3. Subgrade distortion or displacement caused by a lack of load distribution in the overlying pavement. The third type of failure is very common and is the only one related directly to the load transmission studies.

Stress values predicted from the load transmission data may be used in checking theoretical thickness design formulas, in checking the shape and spacing of families of design curves, and in extending empirical design methods to areas not covered adequately by service experience. They also may be used directly in a method of thickness design based on a limiting stress or deflection in the subgrade. For a specific design problem it will be necessary to determine or assume the critical load, tire inflation pressure, and subgrade modulus, and the strength indexes of the materials under consideration for the various pavement layers.

The limiting subgrade deflection for a single load application may vary widely (say from 0.025 to 0.25 in.) for a given design problem, depending primarily upon the expected frequency of load application and the design life of the pavement. Deflection measurements from the WASHO and AASHO test roads, from military traffic tests, and from many universities and highway departments should prove valuable in setting the limit for a specific design condition.

It has been suggested that shear deformation or radius of surface curvature might be a better criterion of impending failure than would vertical subgrade stress or deformation. These suggestions are sound but tend to ignore the practical difficulties of measuring or computing the critical values involved. There certainly is ample evidence from both the field and the laboratory of a relationship between vertical deflection and failure. Evidence of an increasing interest in this relationship is given by the emphasis on deflection measurements of one type or another in the WASHO and AASHO road tests.

Although the triaxial test is the only strength test used extensively for comparing different materials in this CAA program, it is entirely possible that some other test could be used for this purpose. It would be necessary to know (a) that the test could handle the various materials in their normal field condition of moisture and density, (b) that it would simulate to a reasonable degree the field conditions of loading or stress development, and (c) that the measured quality or characteristic would be directly related to the stress resistance or strength of the whole structure.

One point of particular importance in pavement design is the time rate of loading. The critical pavement areas on airports are the aprons and taxi-ways upon which the loads are either static or moving slowly. The situation is somewhat different on highways except in urban areas, and particularly at intersections. All of this simply suggests that the time rate of loading in the basic strength test should be consistent with field loading conditions.

The load transmission test is a static test, corresponding to the most severe condition of airport paving design. The related triaxial tests were run at low rates of loading in order to minimize dynamic effects and provide a fair basis of comparison. Limited tests at other loading rates indicated that the asphaltic concrete was much more sensitive to this variable than were the granular materials. This implies better comparative performance under conditions of short-duration loading, and is consistent with the WASHO traffic tests. It is hoped that this effect can be studied further in subsequent research programs.

FUTURE TEST PROGRAMS

The load transmission testing equipment is now being used in a study of the effects of multiple-wheel loading. This is the only activity scheduled and will terminate the program unless a need is shown for further work of this type.

Although the testing program has been geared specifically to needs of the airport rather than the highway, it appears that the results may be, to some extent, applicable to both. The current AASHO road tests should provide a convenient opportunity to check this assumption.

CONCLUSIONS

1. Data from 804 test loadings have been generalized into curves from which the maximum subgrade reaction can be predicted for a wide range of flexible pavement sections and loading conditions.
2. The generalized data may be used directly in a pavement thickness design method based on a limiting subgrade deflection, or they may be used to supplement or extend other design methods.
3. Although further refinements are possible, particularly in the correlation between load test data and strength tests of materials, the relationships given in this paper are considered accurate enough for practical design use.

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Discussion

W. H. CAMPEN, Manager, Omaha Testing Laboratories, Omaha, Nebraska — Two questions are raised by this paper. First, in addition to reducing pressure on the subgrade, bases must resist consolidation and displacement. In other words, the bases must not displace at all and the consolidation should be neglible. In evaluating the distributive power of the bases in this research, was the deformation in the base courses measured and if so can they be included in the paper?

Secondly, the tests made with the transmission device show that bases composed of different aggregates reduce the pressure on the subgrades at different rates. Did the triaxial tests on the same mixtures show the same relative results? This is brought up because it was suggested that the quality of a base may be measured by the triaxial test.

It should be emphasized that the research conducted on the transmission device is of utmost importance. The research was undertaken at a great expense, to obtain basic information on (a) the distributive power of flexible bases, on subgrades of different strengths, by both tires and rigid steel plates; (b) the effect of quality of base; (c) the effect of composition of base; and (d) the effect of thickness of base and other points.

Although a number of reports have been made on several phases of the research, it is felt that the results have not been fully analyzed and summarized. It is suggested that the final analysis be made by a sub-committee of the Flexible Design Committee of the Highway Research Board.

D. W. LEWIS, Chief Engineer, National Slag Association, Washington, D. C. —This discussion is prompted by the lack of adequate explanation of the aggregate characteristics in the tests which form the basis for the author's summary of the CAA test program results. It is believed that the reader may not be aware of the differences involved, which affect the basis on which valid comparisons could be made.

For example, it is stated in the Synopsis that "the triaxial test is used to compare strengths of the various materials (gravel, clay-gravel, sand, limestone, slag, and asphaltic concrete) used in the pavement sections." Figure 7 shows comparative curves labeled only with the aggregate type—slag, limestone, etc. Inasmuch as slag and limestone are normally furnished in comparable gradings, this would lead the reader to assume that type of aggregate is the major variable involved.

In the section, "Apparatus, Materials, and Procedures," the statement is made that the materials covered "wide ranges of grading, roughness, particle shape, and plasticity: and in Figure 10 the curve for slag bears the label "Macadam Type." However no information is given which would permit the reader to evaluate the actual differences involved. Because this same difficulty is experienced to some extent in previous papers in this series, a review of the characteristics of these materials would appear to be in order.

CAA Technical Development Report No. 144 (author's ref. 2) reports the results of triaxial tests on the various materials used. Figure 1 in this report shows the gradation curves, and indicates that the slag and stone had essentially similar gradings, although the slag contained the lesser amount of fines from the No. 8 sieve down, and had about 1.5 percent of minus No. 200 material compared to 12 percent or more for the stone.

The triaxial results, shown in Tables I-IV and Figures 1-4 in Report No. 144, indicate considerably higher strengths for the slag at all lateral pressures tested. These results led the authors to state in Conclusion No. 7 of the report:

"In load transmission tests of base courses the crushed slag should be the best of the four materials tested triaxially and the sand should be the poorest. This tentative conclusion is subject to verification when load transmission tests are completed on all four materials. This conclusion applies to the particular materials tested, and should not be construed to include all other materials of the same general class."

Because no tests were run with the slag below 5-psi lateral pressure, it is not reasonable to extrapolate the data reported to cross the curve for stone, as is shown in Figure 10 of the current paper.

Reports of the load transmission tests are contained in CAA Technical Development Report No. 269 (author's ref. 6), but using a radically different grading for the slag than had been used in the triaxial tests. Macadam-type construction was used, with 1- to 1½-in. material placed first, followed by fines (all passing the No. 4) vibrated into place. It was described as "a gap-graded material practically devoid of fines passing the No. 200." However, the limestone used was a dense, well-graded crushed aggregate from 1½ in. to 0 with about 10 percent passing the No. 200 sieve, as in the triaxial tests. Thus the variable of grading—well-graded crushed aggregate vs a "macadam grading"—was added to the normal variations in particle shape, surface texture, angle of internal friction, etc., that originally existed between the slag and limestone. However, many of the results are reported or shown graphically in such a manner that the casual reader might well interpret them as showing differences between types of aggregate alone.

Triaxial curves shown in Figure 4 of Report No. 269 are identical to those in Figure 10 of the current paper, with the inference that they are based on data appearing in previous reports. The curves shown for "normal gravel" and "sand" match the average curves in Report No. 144, except that the one for sand has been extended—or extrapolated—to a zero value. The curve shown for limestone correlates closely with the maximum strength values shown in Report No. 144, but the curve for slag appears to be totally different from that previously shown.

The present paper does not resolve this question regarding the basis for the curves shown in Figure 10. If actual triaxial tests with slag in the "macadam grading" have been made, comparison of the results with those on the continuously-graded crushed slag reported previously would provide interesting information on the effects of grading on strength in triaxial compression tests.

With respect to the data as currently presented, however, the writer wishes to emphasize two points as follows:

1. Comparable test results for the two crushed aggregates used—slag and limestone—are available from the CAA studies only in Technical Development Report No. 144, where triaxial test results on similar gradations are reported.
2. Subsequent reports, including this latest summary, contain data based on tests involving the major variable of gradation—in fact, different types of construction—in addition to the variable of aggregate type.

It is unfortunate that these two major variables have been included in this manner. As a result, neither comparisons of slag with limestone (effect of particle shape, etc.), nor of graded crushed aggregate bases with macadam-type bases, can be made with any confidence in their validity.

CLOSURE, Raymond C. Herner--Mr. Campen's kind remarks concerning the load transmission testing program are appreciated and some clarification and elaboration of the specific points raised in his discussion are offered.

In some of the plate loading tests the deflections of both the surface and subgrade were measured. The internal deformation of the base course under load was then determined as the difference between these two deflections. For subgrade deflections in the range from 0.1 to 0.3 in. the ratio of subgrade deflection to surface deflection ranged generally from 65 to 90 percent for 8-in. pavements and from 25 to 50 percent for 24-in. pavements. Corresponding ratios from field test data available from other sources (such as the WASHO test road and the Hybla Valley test track) are of the same general order of magnitude. Surface deflections were not measured in the load transmission tests in which tires were used as the loading medium.

The effect of pavement quality is particularly significant if the pavement rests on a weak subgrade. In such instances the subgrade deflection under a pavement of poor quality may be as much as three times that found under a pavement of high quality and equal thickness. If the pavement is supported on a strong subgrade the effect of pavement quality is less. These comparative effects are shown graphically in Figure 8 of the current paper.

If reasonable allowances are made for accidental and experimental errors, the triaxial test procedure outlined will rate the materials in the same order of performance found in the load transmission tests. Although some unexplained differences have been found, and possible refinement of the correlation process is still being studied, the present procedure is considered to be sufficiently accurate for design purposes.

Up to the present the treatment of load transmission test data by the CAA has consisted primarily of organizing and summarizing the results of the loading tests and the correlation of these results with results from triaxial tests of the paving materials. Although general suggestions have been made for application of the test data to paving design problems, no attempt was made to set up a detailed procedure. If any responsible organization wishes to carry the work further, the CAA certainly will cooperate to the fullest extent possible.

Mr. Lewis indicates that he is interested primarily in a direct comparison of base course materials rather than the over-all objective of the load transmission testing program. He points out that the crushed slag used in the later tests was of somewhat different gradation that that described in T. D. Report No. 144, and infers that this change has tended to show slag to disadvantage in comparison with competing materials. A simple comparison of the reported data is sufficient to disprove this assumption.

Figure 10 of the current paper gives a strength of 107 psi for slag when tested at a lateral pressure of 5 psi, and 177 psi when tested at 10 psi. The corresponding values from Figure 3 of Appendix II of T. D. Report No. 144 are 96 and 139 psi. It appears, therefore, that the change in grading actually has increased the triaxial test values by 10 to 27 percent in the range of lateral pressures for which direct comparisons are available.

This illustrates the inherent danger in making blanket comparisons between competing materials. The CAA tests have shown that the effectiveness of a material for a particular use is governed by its gradation, moisture content, and state of consolidation, as well as by its mineralogical composition. The best material or mixture for one purpose is not necessarily the best for another, and the designer must consider all factors before making his decisions. The load transmission project is concerned only with finding the methods by which the pertinent facts may be measured and evaluated.

Except for the "standard gravel" curve, which is used as a basis of comparison for other materials, the triaxial curves given in the various references are intended only to show the range of values obtainable from materials of widely different physical characteristics. Each pertains only to the specific material tested, and readers have been warned repeatedly against unwarranted generalization of these specific test data. Although a comprehensive triaxial study of materials might well be justified, it is beyond the scope of the current project.

Mr. Lewis is incorrect in assuming that the slag and sand curves were extrapolated to the lower ranges of lateral pressure without benefit of test data. The records show that 46 specimens of the "macadam-type" slag mixture were tested at lateral pressures of less than 5 psi, and these form the basis for the later curves. Because of the voluminous test data from the load transmission project, involving literally millions of individual measurements, the results have been condensed somewhat for publication. The original data are available for inspection by anyone who wishes to avail himself of the opportunity.

Frankly, the author is somewhat mystified by the apparent concern of Mr. Lewis and his associates regarding the tests of their material. All tests to date have shown that slag is an excellent material for base course construction. All reports published thus far—including both T. D. Report No. 144 and the current paper—are entirely consistent on this point.

How Oklahoma Uses Its Present Highways to Establish Criteria for Evaluating Future Structural Design Need

R. A. HELMER, Research Engineer, Oklahoma Department of Highways, Oklahoma City

This paper discusses the possibility of using existing highways as research projects to obtain highway performance data and using existing pedological information to define the soils, topographic and climatic environment of the highways.

It is suggested that analysis of the data from a large number of projects in many different environments would lead to a better understanding of problems of highway design.

● THE PURPOSE of this paper is to call attention to some existing sources of information which are worthy of much serious study by engineers working for the improvement of highway design methods.

Much valuable highway research has consisted of constructing a known kind of road, recording the kind and volume of traffic using it, and evaluating the performance of the paving. To some extent all highways that have been built to a rational design can be used as research projects. Although the records of the design and construction may not have been kept in as much detail as is customary on a research project, the many miles of projects and the variety of conditions and environment available for study will more than compensate for the lack of preliminary data.

The climatic environment of the area in which a research project is constructed is seldom considered as effecting pavement performance, even though there is much evidence to indicate that climate is a major factor in the depreciation of most things; for example, war experience demonstrated that many things which gave good service in the climate of the United States depreciated rapidly in the climate of the Pacific islands.

Soil is so important in the study of pavements that all information about the soils on a research project is unquestionably of value. There is a great store of pedological and agricultural soils knowledge waiting to be put to use by the engineer. The agricultural soils data consist mostly of descriptive information and differ from the usual engineering soils data, which are in the form of figures giving the results of soil tests.

This is an age of specialists. There is a tendency for each group of specialists to confine its thinking to a rather narrow field and to feel that the accumulated knowledge, the multitude of facts, and the many accepted rules developed by the group can only be used within the boundaries of that particular science. It is well to remember that a physical fact or a scientific truth belongs to all groups of people, and can be safely used in any field of study. Thus, translation of pedological ideas and terms to those of engineering may be helpful.

Both the agricultural science and highway engineering begin with the soil. The same rain that erodes a farm will wash away a highway embankment. The same rules that farmers use to reduce erosion of farms will also reduce erosion of highways. The fundamental relationship of agriculture and highway engineering is demonstrated in many ways.

In large areas of good land with a suitable climate, will be found productive farms which will support a large population. So the kind of soil and the climatic environment of the soil, to a large extent, locate the towns. Roads are built to connect towns and it can be said that the soil is a fundamental factor that determines where the farms shall be and also where the roads shall be located, or whether there shall be a road at all. Climate is a major soil-forming factor, as well as a major factor in highway depreciation. When a soil scientist classifies a soil, he also, to a degree, classifies

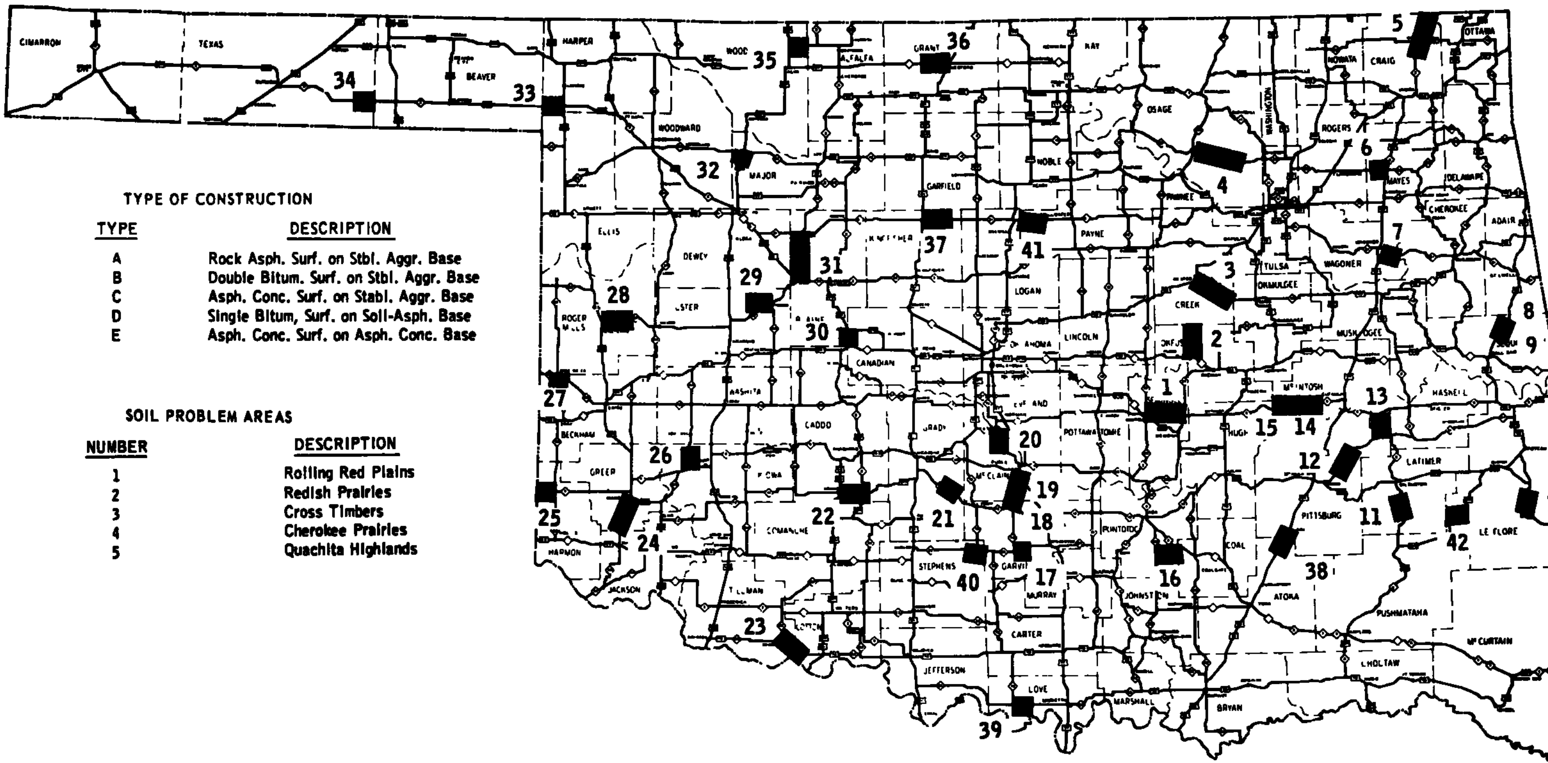


Figure 1. Location of test project sites.

Summary by Type of Construction and Soil Problem Area

PROJECT NUMBER	TYPE OF CONSTRUCTION (Mile) A	B	C	D	E	SOIL PROBLEM AREA (Miles in Each) 1	2	3	4	5	Other
1	11.345							10.345	1.000		
2		10.625						5.625	5.000		
3		14.881						10.000	4.881		
4		15.591						4.591	11.000		
5			13.756						13.756		
6					3.348				3.348		
7		6.101							6.101		
8		5.473								5.473	
9		5.000								5.000	
10		4.920								4.920	
11					5.001					5.001	
12		8.770								8.770	
13	4.600								1.000	3.600	
14					5.053				5.053		
15	10.927							4.500	6.427		
16		4.880						4.880			
17					4.117		3.117	1.000			
18	7.548						7.548				
19	4.677						4.677				
20	7.503						7.503				
21					7.198		7.198				
22		8.794					8.794				
23				8.425		8.425					
24	10.684					10.684					
25	4.808					4.808					
26		4.815				4.815					
27				3.951		3.951					
28				7.568		7.568					
29		6.834				3.834	3.000				
30				5.880			5.880				
31		14.492				12.000		2.492			
32				7.801		7.801					
33				8.000		8.000					
34				6.872		6.872					
35				4.046			4.046				
36				7.989			7.989				
37				8.126			8.126				
38			10.903						10.903		
39				6.166							6.166
40			7.293					5.000			2.293
41			9.120				8.000	1.120			
42			7.630							7.630	
SUMMARY ALL PROJECTS	62.092					15.492	19.728	14.845	8.427	3.600	
		111.176				20.649	11.794	27.588	26.982	24.163	
			48.702				8.000	6.120	24.659	7.630	2.293
				74.824		42.617	26.041				6.166
					24.717		10.315	1.000	8.401	5.001	
SUBTOTALS	62.092	111.176	48.702	74.824	24.717	78.758	75.878	49.553	68.469	40.394	8.459
GRAND TOTALS	All Types: 321.511 Miles					All Areas: 321.511 Miles					

HIGHWAY ENGINEERING CHARACTERISTICS OF SOIL SERIES

SOIL SERIES and underlying GEOLOGIC FORMATIONS	SOIL ASSOCIATION NUMBER	MAPPED IN DIVISION NO. (ASSOC. MAP)	MAPPING UNIT	RATING NUMBER	BASE REQ'D FOR 9,000# WHEEL LOAD (by Group Index Method)	TEXTURE	PERMEABILITY	COLOR	B.P.R. SOIL CLASSIFICATION	LIQUID LIMIT	PLASTIC INDEX	% PASSING NO.200 SIEVE	CALIFORNIA BEARING RATIO	SHRINKAGE LIMIT	FIELD MOISTURE EQUIVALENT	VOLUMETRIC SHRINKAGE	FOR STABILIZATION		LOAD SUPPORTING ABILITY			VOLUMETRIC SHRINKAGE			SOIL BINDER		SOIL STABILIZATION		SHOULDER CONSTRUCTION			EROSION CONTROL			SEEDING AND SODDING		
																	% ASPHALT	% PORTLAND CEMENT	G	F	P	H	M	L	Y	N	Y	N	G	F	P	G	F	P	G	F	P
BERTHOUD	51	6	7																																		
"A" 0-14"					9"	M	M	GB	A-4(4)	27	9	54	7				7.0				X			X		X		X			X			X		X	
"B" 14-30"					12"	MF	M	GB	A-6(9)	33	12	73	8				NO				X			X		X		X			X			X		X	
"C" 30-60"					10"	MF	M	GB,W	A-6(8)	25	8	83	7				NO				X			X		X		X			X			X		X	
"D" 60"+																																					
Dakota ss																																					
Colorado grp																																					
Purgatoire fm																																					
Morrison fm																																					
Dockum fm					15"				A-7-6(14)	49	20	84	4				NO				X	X				X		X			X		X			X	
BROWNFIELD	42	5,6	12																																		
"A" 0-25"					6"	C	MR	RB	A-2(0)	18	1	33	14				5.0				X			X		X	X				X			X		X	
"B" 25-50"					9"	MF	MR	R	A-6(2)	30	11	44	13				6.0				X			X	X			X		X				X	X		
"C" 50-70"					11"	M	MR	YR	A-6(8)	34	14	63	5				NO				X		X			X		X			X			X			X
"D" 70"+																																					
Tertiary Agefms					10"				A-2(0)	30	10	56	28				7.0				X			X		X		X			X			X		X	
Terrace deposits					9"				A-4(8)	26	3	92	8				NO				X			X		X		X			X			X			X
CAREY	38,38-A	4,5,6,7	7																																		
"A" 0-10"					10"	M	MR	RB	A-4(8)	28	5	90	4				NO				X			X		X		X			X			X		X	
"B" 10-40"					13"	MF	MR	R,RB	A-6(10)	39	14	85	6				NO				X		X			X		X			X			X		X	
"C" 40-60"					12"	M	MR	R	A-6(9)	30	12	78	9				NO				X			X		X		X			X			X		X	
"D" 60"+																																					
Cloud Chief fm					15"				A-7-6(13)	47	21	97	3				NO				X	X				X		X			X	X					X
Rush Springs ss					9"				A-4(4)	30	4	55	18				6.5				X			X		X		X			X			X		X	
Marlow fm					13"				A-7-6(10)	41	16	97	5				NO				X	X				X		X			X		X				X
Quartermaster fm					11"				A-4(8)	31	10	89	33				NO				X			X		X		X			X			X		X	
Dog Creek shale					9"				A-4(8)	23	2	94	4				NO				X			X		X		X			X			X			X
COTTONWOOD	36,37	4,5,6,7	25																																		
"A" 0-8"					10"	MF	M	GB	A-4(8)	27	4	87	9				NO				X			X		X		X			X			X		X	
"C" 8-40"+					10"			W	A-4(7)	30	4	73	8				NO				X			X		X		X			X			X		X	
Blaine gyp					14"				A-7-6(12)	43	19	97	5				NO				X	X				X		X			X		X				X
Flowerpot shale					12"				A-4(8)	37	10	92	11				NO				X		X			X		X			X			X			X
Cloud Chief fm					15"				A-7-6(13)	47	21	97	3				NO				X	X				X		X			X	X					X
DALHART	48,50	6	12																																		
"A" 0-8"					6"	MC	M	YB	A-2-3(0)	NP	NP	14	13				4.0			X				X		X		X			X			X			X
"B" 8-33"					8"	MF	M	B	A-4(1)	29	10	42	10				6.0				X			X	X			X		X				X	X		
"Cca"33-54"					11"	F	M	B	A-6(7)	35	15	62	5				NO				X		X			X		X			X		X			X	
"C" 54"+																																					
Tertiary Age fms					6"				A-2-3(0)	NP	NP	10	32				NO			X				X		X		X			X			X			X
Terrace deposits					9"				A-4(8)	26	3	92	8				NO				X			X		X		X			X			X			X

ENTERPRISE	44	5,6,7	7X																																		
"A" 0-15"					6"	MC	MR	RB	A-2-3(0)	NP	NP	13	4				NO				X			X		X		X			X			X			X
"C" 15-50"					6"	MC	MR	RB	A-3(0)	NP	NP	9	5				NO			X				X		X		X			X			X			X
"D" 50"/																																					
Terrace deposit					9"				A-4(8)	26	3	92	8				NO				X			X		X		X			X		X				X
Tertiary Age fms					11"				A-2-3(0)	30	10	56	28				7.0				X			X		X		X			X			X		X	
Dog Creek shale					14"				A-6(11)	39	18	84	5				NO				X		X			X		X			X		X			X	
Dune sand					6"				A-2(0)	17	1	17	12				4.0			X				X		X	X				X			X		X	
Alluvium					10"				A-4(8)	27	6	77	9				NO				X			X		X		X			X			X		X	
GRANT	31	4,6	7																																		
"A" 0-20"					9"	MF	MR	B	A-4(1)	23	3	41	28				5.5				X			X		X	X			X				X		X	
"B" 20-48"					6"	MF	MR	RB,YR	A-4(2)	23	1	45	16				5.5				X			X		X	X				X			X		X	
"C" 48-72"					10"	MF	MR	YR	A-4(8)	29	5	85	12				NO				X			X		X		X			X			X		X	
"D" 72"/																																					
Cedar Hills ss																																					
Hennessey shale					14"				A-7-6(12)	43	20	94	9				NO				X	X				X		X			X		X				X
Flowerpot shale					12"				A-4(8)	37	10	92	11				NO				X		X			X		X			X			X			X
KIRKLAND	28,29	3,4,5,6,7,8	5																																		
"A" 0-11"					10"	M	VS	B	A-4(8)	28	6	94	4				NO				X			X		X		X			X			X			X
"B" 11-38"					19"	F	VS	B	A-7-6(19)	53	30	95	9				NO				X	X				X		X			X	X					X
"C" 38-100"					15"	F	VS	RB	A-7-6(15)	49	23	98	4				NO				X	X				X		X			X	X					X
"D" 100"/																																					
Hennessey shale					16"				A-7-6(16)	47	26	95	7				NO				X	X				X		X			X	X					X
Garber ss					13"				A-6(6)	38	13	61	10				NO				X		X			X		X			X			X		X	
Wichita fm					12"				A-6(9)	33	13	96	5				NO				X		X			X		X			X			X			X
Wellington fm					17"				A-7-6(18)	50	28	93	4				NO				X	X				X		X			X	X					X
Flowerpot shale					12"				A-4(8)	37	10	92	11				NO				X		X			X		X			X			X			X
Upper Pontotoc grp																																					
LELA	7	ALL	3																																		
"A" 0-45"					14"	F	VS	G	A-7-6(13)	42	20	90	5				NO				X	X				X		X			X		X			X	
"AC" 45-75"						F	VS	RB																													
"C" 75-90"						F	VS	RB																													
"D" 90"/																																					
Alluvium					16"				A-7-6(15)	50	23	90	5				NO				X	X				X		X			X			X		X	
LINCOLN	6	ALL	15																																		
"A" 0-15"					9"	C	R	B	A-4(5)	24	4	61	4				NO				X	X				X		X			X			X		X	
"C" 15-60"/					6"	C	R	YB	A-2-3(0)	NP	NP	17	13				NO			X				X		X	X				X			X			X
Alluvium					10"				A-4(8)	28	3	81	12				NO				X	X				X		X			X			X		X	
MANSKER	47	6,	18																																		
"A"0-15"					8"	MF	M	GB	A-4(4)	22	5	45	10				5.5				X			X		X	X			X				X		X	
"Cca" 15-40"					12"	MF	M	B	A-6(7)	35	11	71	9				NO				X		X			X		X			X			X		X	
"C" 40-60"					10"	MF	M	B	A-4(4)	28	10	57	3				7.0				X			X		X		X			X			X		X	
"D" 60"/																																					
Tertiary Age fms					5"				A-7-6(13)	49	18	76	3				NO				X	X				X		X			X		X			X	
Cloud Chief fm																																					
Dakota ss																																					
Morrison fm																																					
MILLER	1,6	ALL	3																																		
"A" 0-8"					12"	F	VS	RB	A-6(8)	32	11	93	4				NO				X			X		X		X			X			X			X
"C" 8-46"					13"	F	VS	RB	A-6(10)	39	14	98	5				NO				X		X			X		X			X			X			X
"D" 46"/																																					
Alluvium					16"				A-7-6(15)	50	23	90	5				NO				X	X				X		X			X	X				X	

HIGHWAY ENGINEERING CHARACTERISTICS OF SOIL SERIES

SOIL SERIES and underlying GEOLOGIC FORMATIONS	SOIL ASSOCIATION NUMBER	MAPPED IN DIVISION NO (ASSOC. MAP)	MAPPING UNIT	RATING NUMBER	BASE REQ'D FOR 9,000# WHEEL LOAD (by Group Index Method)	TEXTURE	PERMEABILITY	COLOR	B P R SOIL CLASSIFICATION	LIQUID LIMIT	PLASTIC INDEX	% PASSING NO 200 SIEVE	CALIFORNIA BEARING RATIO	SHRINKAGE LIMIT	FIELD MOISTURE EQUIVALENT	VOLUMETRIC SHRINKAGE	FOR STABILIZATION: % ASPHALT	FOR STABILIZATION: % PORTLAND CEMENT
NASH	31	4,6	7															
"A" 0-6"					8"	M	MR	RB	A-4(3)	24	3	52	22				6.0	
"B" 6-26"					8"	M	MR	YR	A-4(2)	27	2	46	29				5.5	
"C" 26-36"+																		
Cedar Hills ss																		
Hennessey shale					13"				A-6(10)	36	14	76	9				NO	
Flowerpot shale																		
POND CREEK	31	4,6	6															
"A" 0-15"					10"	M	M	B	A-4(8)	30	7	80	9				NO	
"B" 15-36"					13"	MF	M	B	A-6(9)	37	13	81	8				NO	
"C" 36-72"					8"	MF	M	B,R	A-4(2)	24	5	48	9				6.0	
"D" 72"+																		
Cedar Hills ss																		
Hennessey shale					13"				A-6(10)	36	14	76	9				NO	
Flowerpot shale																		
PORT	7	ALL	4															
"A" 0-14"					9"	MF	M	B	A-4(7)	25	6	70	5				NO	
"AC" 14-30"					10"	MF	M	B	A-4(8)	27	8	77	3				NO	
"C" 30-60"+ Alluvium					16"	MF	M	RB	A-7-6(15)	50	23	90	5				NO	
POTTER	47	6	25															
"A" 0-8"					6"	M	M	GB	A-2-4(0)	34	7	21	23				NO	
"C" 8"+					7"				A-4(2)	22	3	45	31				5.5	
Tertiary Age fms					6"				A-2-3(0)	NP	NP	10	37				NO	
Cloud Chief fm																		
PRATT	40	4,5,6	12X															
"A" 0-12"					6"	MC	MR	B	A-2(0)	21	3	34	17				5.0	
"B" 12-26"					6"	MF	MR	B,YR	A-4(1)	25	6	30	22				5.0	
"C" 26-40"					6"	MC	MR	B	A-2(0)	20	3	26	11				4.5	
"D" 40"+																		
Terrace deposits					9"				A-4(8)	26	3	92	8				NO	
Tertiary Age fms					6"				A-2-3(0)	NP	NP	10	32				NO	
Dune sand					6"				A-2(0)	17	1	17	12				4.0	
PULASKI	7	ALL	9															
"A" 0-18"					6"	M	MR	B	A-2-3(0)	NP	NP	31	18				5.0	
"C" 18-50"+					6"	M	MR	B	A-4(1)	21	1	41	21				5.5	
Alluvium					10"				A-4(8)	26	5	81	15				NO	

SOIL SERIES	LOAD SUPPORTING ABILITY			VOLUMETRIC SHRINKAGE			SOIL BINDER		SOIL STABILIZATION		SHOULDER CONSTRUCTION			EROSION CONTROL			SEEDING AND SODDING		
	G	F	P	H	M	L	Y	N	Y	N	G	F	P	G	F	P	G	F	P
NASH "A" 0-6"			X			X		X		X			X			X		X	
"B" 6-26"			X			X		X	X				X			X		X	
Hennessey shale			X		X			X		X			X			X		X	
POND CREEK "A" 0-15"			X			X		X		X			X			X		X	
"B" 15-36"			X		X			X		X			X			X		X	
"C" 36-72"			X			X		X	X			X				X		X	
Hennessey shale			X		X			X		X			X	X				X	
PORT "A" 0-14"			X			X		X		X			X			X		X	
"AC" 14-30"			X			X		X		X			X			X		X	
"C" 30-60"+ Alluvium			X	X				X		X			X	X				X	
POTTER "A" 0-8"	X					X		X	X		X					X	X		
"C" 8"+		X				X		X	X			X				X		X	
Tertiary Age fms	X					X		X	X				X			X			X
PRATT "A" 0-12"			X			X		X	X		X					X		X	
"B" 12-26"	X					X		X	X		X					X	X		
"C" 26-40"	X					X		X	X		X					X		X	
Terrace deposits			X			X		X		X			X			X			X
Tertiary Age fms	X					X		X		X			X			X			X
Dune sand	X					X		X	X				X			X		X	
PULASKI "A" 0-18"	X					X		X	X				X			X			X
"C" 18-50"+			X			X		X	X				X			X		X	
Alluvium			X			X		X		X			X			X		X	

PULLMAN	49	6	2																																		
"A" 0-5"						MP	S	GB										12																			
"B" 5-40"						F	S	B										12																			
"C" 40-60"						F	S	RY										12																			
"D" 60"+																																					
Tertiary Age fms																																					
QUINLAN	32,38,39	4,5,6,7	20																																		
"A" 0-15"					6"	M	R	RB	A-2(0)	26	2	34	12				5.0			X				X		X	X				X				X	X	
"C" 15"+					6"	C	R	R	A-2(0)	26	2	40	19				5.5				X			X		X	X				X				X	X	
Rush Springs ss					8"				A-4(4)	30	4	55	18				6.5				X			X		X		X			X				X	X	
Cloud Chief fm					9"				A-4(8)	23	1	90	11				NO				X			X		X		X			X				X	X	
Quartermaster fm																																					
Marlow fm					11"				A-4(8)	33	10	88	5				NO				X			X		X		X			X				X	X	
Flowerpot shale																																					
RENFROW	28,30	3,4,5,6,7,8	1																																		
"A" 0-10"					10"	MP	VS	RB	A-4(7)	26	4	70	3				NO				X			X		X		X			X				X	X	
"B" 10-26"					14"	F	VS	RB	A-7-6(12)	43	19	84	10				NO				X	X				X		X			X		X			X	
"C" 26-50"					18"	F	VS	R	A-7-6(19)	53	29	87	10				NO				X	X				X		X			X	X					X
"D" 50"+																																					
Hennessey shale					16"				A-7-6(16)	47	26	95	7				NO				X	X				X		X			X	X					X
Garber ss					12"				A-6(6)	38	13	61	10				NO				X		X			X		X			X				X	X	
Wichita fm					12"				A-6(9)	33	13	96	5				NO				X		X			X		X			X				X		X
Upper Pontotoc grp																																					
Wellington fm					17"				A-7-6(18)	50	28	93	3				NO				X	X				X		X			X	X					X
Flowerpot shale					12"				A-4(8)	37	10	92	11				NO				X		X			X		X			X				X		X
RICHFIELD	48,49	6	2																																		
"A" 0-13"					13"	M	M	GB	A-6(10)	36	16	82	2				NO	10			X		X			X		X			X		X			X	
"B" 13-35"					16"	MP	M	B	A-7-6(15)	48	23	89	4				NO	12			X	X				X		X			X	X				X	
"C" 35-130"					12"	MP	M	B,W,P	A-6(9)	34	12	74	5				NO	12			X		X			X		X			X			X		X	
"D" 130"+																																					
Tertiary Age fms																																					
Terrace deposits					9"				A-4(8)	26	3	92	8				NO				X			X		X		X			X			X			X
ST. PAUL	38-A	5,6	6																																		
"A" 0-20"					12"	MP	M	B	A-4(8)	33	10	82	5				NO	10			X			X		X		X			X			X		X	
"B" 20-60"					13"	F	M	RY	A-6(10)	40	16	74	9				NO	10			X		X			X		X			X		X			X	
"C" 60-90"					14"	F	M	RY	A-6(12)	40	21	87	5				NO	10			X	X				X		X			X	X				X	
"D" 90"+																																					
Rush Springs ss																																					
Cloud Chief fm					11"				A-4(8)	30	10	83	8				NO				X			X		X		X			X			X		X	
TIPTON	44	5,6,7	9H																																		
"A" 0-35"					10"	MP	M	B	A-4(8)	26	4	85	11				NO				X			X		X		X			X			X		X	
"B" 35-70"					13"	MP	M	RB,RY	A-6(11)	37	16	81	5				NO				X		X			X		X			X		X			X	
"C" 70-90"					13"	MP	M	RY	A-6(11)	36	18	89	6				NO				X		X			X		X			X		X			X	
"D" 90"+																																					
Terrace deposits																																					
Tertiary Age fms																																					
Dog Creek shale					14"				A-6(11)	39	18	86	5				NO				X		X			X		X			X		X			X	
Alluvium					10"				A-4(8)	27	6	77	9				NO				X			X		X		X			X			X		X	
TIVOLI	41,42	4,5,6	13																																		
"A" 0-8"					6"	C	R	B	A-3(0)	NP	NP	8	9				NO	10		X				X		X		X			X			X			X
"C" 8-60"					6"	C	R	Y	A-3(0)	NP	NP	3	10				NO	10		X				X		X		X			X			X			X
"D" 60"+																																					
Dune sand					6"				A-3(0)	NP	NP	8	10				NO			X				X		X		X			X			X			X
Terrace deposits					6"				A-2-3(0)	NP	NP	16	11				4.0			X				X		X	X				X			X			X
Tertiary Age fms					6"				A-2-3(0)	NP	NP	10	32				NO			X				X		X		X			X			X			X

HIGHWAY ENGINEERING CHARACTERISTICS OF SOIL SERIES

SOIL SERIES and underlying GEOLOGIC FORMATIONS	SOIL ASSOCIATION NUMBER	MAPPED IN DIVISION NO. (ASSOC. MAP)	MAPPING UNIT	RATING NUMBER	BASE REQ'D FOR 9,000# WHEEL LOAD (by Group Index Method)	TEXTURE	PERMEABILITY	COLOR	B.P.R. SOIL CLASSIFICATION	LIQUID LIMIT	PLASTIC INDEX	% PASSING NO.200 SIEVE	CALIFORNIA BEARING RATIO	SHRINKAGE LIMIT	FIELD MOISTURE EQUIVALENT	VOLUMETRIC SHRINKAGE	FOR STABILIZATION % ASPHALT	FOR STABILIZATION % PORTLAND CEMENT	LOAD SUPPORTING ABILITY G	LOAD SUPPORTING ABILITY F	LOAD SUPPORTING ABILITY P	VOLUMETRIC SHRINKAGE H	VOLUMETRIC SHRINKAGE M	VOLUMETRIC SHRINKAGE L	SOIL BINDER Y	SOIL BINDER N	SOIL STABILIZATION Y	SOIL STABILIZATION N	SHOULDER CONSTRUCTION G	SHOULDER CONSTRUCTION F	SHOULDER CONSTRUCTION P	EROSION CONTROL G	EROSION CONTROL F	EROSION CONTROL P	SEEDING AND SODDING G	SEEDING AND SODDING F	SEEDING AND SODDING P
TRAVESSILLA "A" 0-8" "C" 8"/ Dakota ss Colorado grp Purgatoire grp Morrison fm Dockum fm	51	6	20		9"	M	M	B	A-4(3)	27	9	52	2				6.5				X			X		X		X			X			X		X	
VERNON "A" 0-8" "C" 8-15" "D" 50"/ Hennessey shale Wichita fm Upper Pontotoc grp Garber ss Wellington fm Flowerpot sh	28,34	3,4,5,6,7,8	17		12"	F	VS	B	A-6(8)	34	11	73	6				NO				X			X		X		X			X			X		X	
					14"	F	VS	RB	A-7-6(12)	44	18	87	6				NO				X	X				X		X			X		X			X	
					16"				A-7-6(16)	47	26	95	7				NO				X	X				X		X			X	X					X
					12"				A-6(9)	33	13	96	5				NO				X		X			X		X			X			X			X
					11"				A-6(6)	38	13	61	10				NO				X		X			X		X			X			X		X	
					17"				A-7-6(18)	50	28	93	4				NO				X	X				X		X			X	X					X
VONA "A" 0-8" "B" 8-36" "C" 36-50" "D" 50"/ Tertiary Age fms Terrace deposits Dune sand	50	6	7X		6"	MC MF	R R	B B	A-2(0)	20	1	16	6				NO			X				X		X	X				X			X		X	
					6"	C	R	B	A-3(0)	NP	NP	5	9				NO			X				X		X		X			X			X			X
					6"				A-2-3(0)	NP	NP	10	32				NO			X				X		X		X			X			X			X
					6"				A-2-3(0)	NP	NP	10	16				NO			X				X		X		X			X			X			X
WOODWARD "A" 0-20" "Cca" 20-26" "C" 26"/ Cloud Chief fm Rush Springs ss Marlow fm Dog Creek shale	38	4,5,6,7	20		10"	M	MR	RB	A-4(8)	27	5	77	4				NO				X			X		X		X			X			X		X	
					11"	M	MR	R	A-4(8)	30	8	89	7				NO				X			X		X		X			X			X		X	
					11"	M	MR	R	A-4(7)	33	9	72	5				NO				X			X		X		X			X			X		X	
					11"				A-4(8)	30	10	83	8				NO				X			X		X		X			X			X		X	
					9"				A-4(3)	28	4	68	13				NO				X			X		X		X			X			X		X	
					12"				A-4(8)	33	10	82	8				NO				X			X		X		X			X			X		X	
					9"				A-4(8)	23	2	94	4				NO				X			X		X		X			X			X			X
YAHOLA "A" 0-18" "C" 18-60" "D" 60"/ Alluvium	1,4,6	ALL	9		10"	M	MR	RB	A-4(6)	22	6	63	8				NO				X			X		X		X			X			X		X	
					6"	M	R	RB	A-2-3(0)	NP	NP	30	13				5.0			X				X		X	X				X			X			X
					11"				A-4(8)	29	7	95	9				NO				X			X		X		X			X			X			X

Highways, like rocks, are subject to damage by water, by wetting and drying, by expansion and contraction from heat and cold, and by the powerful forces of freezing moisture. The damage over a period of time will be large or small depending on the magnitude of the destructive factors at the particular location. If, by determining the name of the soil series, the subgrade soils are defined and the materials and climatic conditions that produce the soil are indicated, some of the factors which determine the behavior of the paving are also known. By using past experience, the name of the soil series might indicate the condition of the road that one would expect to find in areas where the soil is used as subgrade along the highway.

The records of the Oklahoma research project give the names of not many more than 100 soil series in 321 miles of paving. It is estimated that these 100 principal soils cover about 75 percent of the area of Oklahoma. When the topsoil, the subsoil, and the underlying geology are each counted as a kind of engineering soil, there are then 300 kinds of soil indicated by the 100 different series. If it is assumed that one-half of the 300 miles of highway in the research project is in fill, there are about 150 miles in cut, and it is indicated that the average soil extent for each of the 300 engineering soils is something less than one-half mile. Therefore, for the roads covered by the pedological soil survey of the project, there exists for each half-mile of paving a record that gives not only the thickness and texture of each soil horizon, but also the climate, topography, the internal drainage of the soil, the name or description of the underlying geology, and the natural vegetation in the area. The highway records for these projects have been investigated, the condition of the paving recorded, the depreciation of these roads calculated, and the information classified by soil series name.

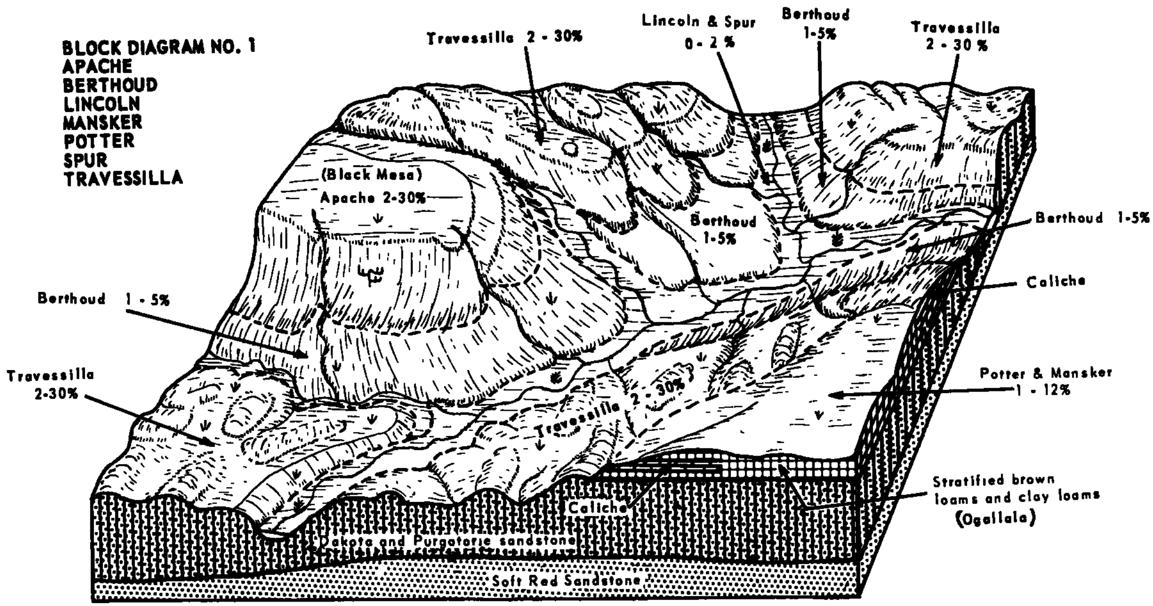
An attempt has been made to organize the wealth of published pedological soil data and develop methods of interpretation so it can be used to identify soils by inspection and examination in the field and to evaluate the principal Oklahoma soils for engineering uses. Laboratory tests of the soil and geology samples have been recorded by the name and horizon of the soil series and the name of the geologic formation. These tests have been interpreted for highway use and an estimate has been made of the suitability of the materials for the following common uses that are made of soils in highway work: subgrade, shoulders, erosion control, seeding and sodding, shrinkage, soil binder, and soil stabilization. Also by correlating the soil series name with the records of highway experience, there have been assigned to each soil series such detailed information as required base thickness, load supporting value, and the depreciation of various kinds of paving constructed on it.

For field identification, a policy has been adopted of dividing the soils of the state into groups of principal soils by location; for example, the soils found in each individual maintenance division. By considering each of the eight divisions separately, the number of principal soils to be recognized is reduced from 94 for the state to approximately 30 in each division. With only 30 soils to classify, the problem of classification and identification in the field is greatly simplified.

The U. S. Soil Conservation Service has divided the state into a number of problem areas. These areas are large; one may extend into several states, so only a few of them are found in any one maintenance division. The descriptions for these large soil problem areas are very general. The areas are defined by climate, underlying geological material, vegetation, and topography. The study has suggested that soils are not the basic cause of highway problems but, like the soils themselves these problems are the result of the conditions of climate, geology, topography and traffic prevailing in certain areas of the state and the problems of highways are problems of the environment in which the highway is constructed. These large areas of similar environment can be thought of as areas of similar highway problems. The problem areas have been divided into soil associations and the soil association areas can be divided into soil series areas.

This research project can be thought of as the installation of a bookkeeping system to record, by soil series name, all soils and highway information as it becomes known and to make it available for future use. As time goes on and the records include more and more material, their value will increase.

It has been said that the kind of soil present at a particular place is determined by



Soils of Northwest Cimarron County, Oklahoma developed in sandstones, shales and in basalts, (Black Mesa) Remmants of Ogallala loams and clay loams in right front corner. Caliche outcrops along "Breaks".

ORIGINAL VEGETATION

Short Grass Tall Grass Pinon - Juniper

Figure 2. Typical block diagram for soils and geology.

five principal soil-forming factors, which are the action of climate and living organisms (such as plant and animal life) upon the parent material (such as geological strata) or reworked material (such as alluvium) modified by local relief and the time during which the action has been going on. This definition implies that the type of local relief is more or less uniform for a given soil series and also for each soil series the underlying geology is of a similar kind. Because this is true, a typical environment of topography and geology can be shown for each soil series by a diagram.

Block diagrams which show the typical topographic environment and underlying geology of the principal soil series within each soil association can be used to divide the soil association areas when maps which show the soil areas occupied by each soil series are not available. These block diagrams give a detailed picture of where each principal soil series in a soil association may be found and the location of some of the minor soils found associated with the principal soils (see Fig. 2).

The steps of classification of the soils of Oklahoma by maps are as follows:

1. Maintenance division.
2. Problem area.
3. Soil association
4. Block diagram.

The soil series have been mapped in detail by the Department of Agriculture for many counties of the state. If these maps are available, the location of each individual soil series has already been determined and the name of the soil series at any location can be obtained by finding the particular location on the map. The engineering data about a principal soil can be found in the State Highway Department records. An estimate for a minor soil can be made by comparing it with a similar principal soil.

When using county soil maps, the block diagrams help the engineer to recognize a soil in the field by indicating the underlying geology and the kind of topography on which the soil will be developed.

Engineering soil information is obtained for many purposes. The degree of detail to which it is necessary to define the soil depends on the purpose for which it is intended to use the information. At times, the name of only the soil problem area or soil association may give all the information necessary.

The system of filing the records of highway behavior by the soil series name is very new. In fact, it has not yet been fully developed. But already there are examples of failures in a highway that can be analyzed by referring to the soil series descriptions. The road was built in 1949 and gave excellent service for about seven years. In 1956 it was in excellent condition; a detailed condition survey gave this road a rating of 99 percent perfect. In 1957, the rains came in a record-breaking quantity. A condition survey afterward gave the highway a rating of 84 percent perfect, a loss of 15 percent in a year or less. This 14-mile long project was built at a cost (1950 prices) of about $23,500 per mile, an investment of something more than $329,000 in the paving of this project. If the damage in one year is 15 percent, the loss was in the neighborhood of $50,000 on this 14 miles of paving. This road is constructed in a area of Dennis, Bates, and Parsons soils. The second condition survey indicated that the damaged areas occurred in the transition zone where the soils changed from Dennis or Bates to the Parsons soil. The typical pedological descriptions of these soils give the information that Dennis and Bates soils are on steeper slopes than Parsons soils. The surface runoff of these soils is indicated to be slow to moderately rapid. The internal drainage is moderate to moderately slow and silty and sandy clay loams are found at depths of from 10 to 22 inches. On the other hand, a Parsons soil is described as being on nearly level land with a runoff that is medium to low. The internal drainage is indicated to be slow to very slow, with a claypan at a depth of 12 to 18 inches.

With this general information about the soils it is easy to visualize the development of these failures. This road was built and put in service during a period of below normal rainfall. When sufficient water was present and the soils became saturated, the more permeable soils on the steeper slopes and the higher elevations gradually drained downgrade to the transition zone between these soils and the Parsons soil, which was in the flats and had a claypan to block further drainage. This condition resulted in an accumulation of water in the transition zone. The claypan of the Parsons soils and the clay loam soils of the Dennis and Bates soils became waterlogged. Being saturated with water, the subgrade soil beneath the paving lost its cohesive strength and could not support the weight of the traffic on the highway and failure resulted. From the engineering tests of the soils at the time this project was constructed, it was impossible to predict or visualize a condition of internal drainage that would develop seven years in the future.

With this knowledge available for future use on other projects, whether the wet condition is apparent or not at the time of construction, it will be known that these possible failure areas should be protected. This kind of failure can be prevented by placing subdrains to carry off the accumulated moisture at the transition zones between the more pervious soils on the steeper slopes and the Parsons soils lying on the flats below them.

EXAMPLE OF USE OF RECORDS

Highways are located by control points, such as towns, and crossings of rivers and railroads. The problem is to find the best and cheapest route between two points.

US 70 in Oklahoma was located long ago. It crosses Choctaw County from east to west, a distance of 45 miles. A soil survey of this county published in 1943 showed that about 13 miles of this highway are in an area which has principally sandy loams of the Bowie, Cuthbert, Kirvin, Sawyer, and other similar soils. Roughly, the average thickness of paving for a highway on these soils is 15 inches of base and subbase. The remaining 32 miles are in an area having principally such plastic soils as Choctaw, Durant, Denton clay, and San Saba clay. The required thickness of base and subbase for these plastic soils is much greater than for the sandy loams and an average thickness of about 26 inches would be required. This is a difference of about 11 inches for the 32 miles of heavy soils. Using a 1950 price index, an inch of paving costs from $1,800 to $2,000 per mile, or a total of $700,000 for the 32 miles. The soil map indicates that by moving the highway location not much more than a mile the clay areas could have been avoided. If the survey had been available at the time the road was located, and if this information had been used, the mapping of the soils in Choctaw County could have been worth $700,000 to the people who pay the tax on gasoline.

Analysis of Flexible Pavement Deflection and Behavior Data

A. C. BENKELMAN, Flexible Pavement Research Engineer, AASHO Road Test

In the WASHO Road Test there was a pronounced difference in the performance of the edge and center portions of the bituminous pavement, as well as a great difference in behavior of sections with 2 in. of surfacing and those with 4 in. of surfacing.

This paper presents a series of relations to indicate to what extent the over-all pavement structure thickness and the seasons of the year enter into the differences in behavior of the edge and center portions of the pavement, and into the difference in behavior of the sections with 2-in. and 4-in. surfacing.

● IT HAS LONG been a matter of common knowledge that the edges of a pavement of the flexible type are more likely to fail under traffic than the interior portions.

The WASHO Road Test served to provide numerical values as to the degree of difference that might be expected in the behavior of the edge and center areas of this type of pavement. For example, at the end of this test almost twice as much distress had developed along the edge as in the interior. In the three thicker sections of pavement in the test the distress that developed was practically all found to be along the edge.

The findings of the WASHO test in regard to this particular problem and what might be done about it in the design and construction of new pavement have received the attention of highway engineers in all sections of the country. One possible solution that was investigated to a limited extent at the project site in Idaho involves paving of the shoulders. Soon after test traffic was started failures began to develop along the edge of the pavement and it was decided to pave the shoulders of part of some of the sections and to observe if this would prove effective in retarding the rate of development of distress of the original edge of pavement. Before the end of the test it was concluded that to all intents and purposes the outer wheel path (OWP) with paved shoulders became the equivalent of the inner wheel path (IWP) of travel. In this connection it should be pointed out that the observed difference in behavior of the test pavement in the outer and inner wheel paths where the shoulders were not paved was more pronounced than that between two adjacent sections where the difference in over-all structure thickness was 4 in. This would seem to indicate that if the shoulders of such pavements are paved some reduction in the over-all thickness of the structure would be justified. However, this would require some positive means by which the movement of traffic could be controlled to prevent operation of heavy loads on the shoulders.

As shown in Figure 1, the WASHO pavement consisted of two identical test loops. One tangent of each of the loops included five 300-ft test sections having a 2-in. asphaltic concrete surface and a 4-in. base course; one of the sections was laid directly on the selected embankment subgrade soil, the other four on an intervening subbase course, 4, 8, 12, and 16 in. thick. The other tangent of each of the loops contained identical sections except that the surface course was 4 in. and the base course 2 in. thick. Single-axle loads (18,000 and 22,400 lb) operated on each 12-ft lane of one of the loops and tandem-axle loads (32,000 and 40,000 lb) operated on each lane of the other loop.

As stated in Part 2 of the WASHO Road Test report (HRB Special Report 22) the structural distress that developed in the test pavement was confined largely to two critical periods of traffic operation. Because this was the case, data were assembled in the report (Table 4-d-1) relative to the observed behavior and deflection of the pavement for certain designated periods of the year. These periods were as follows:

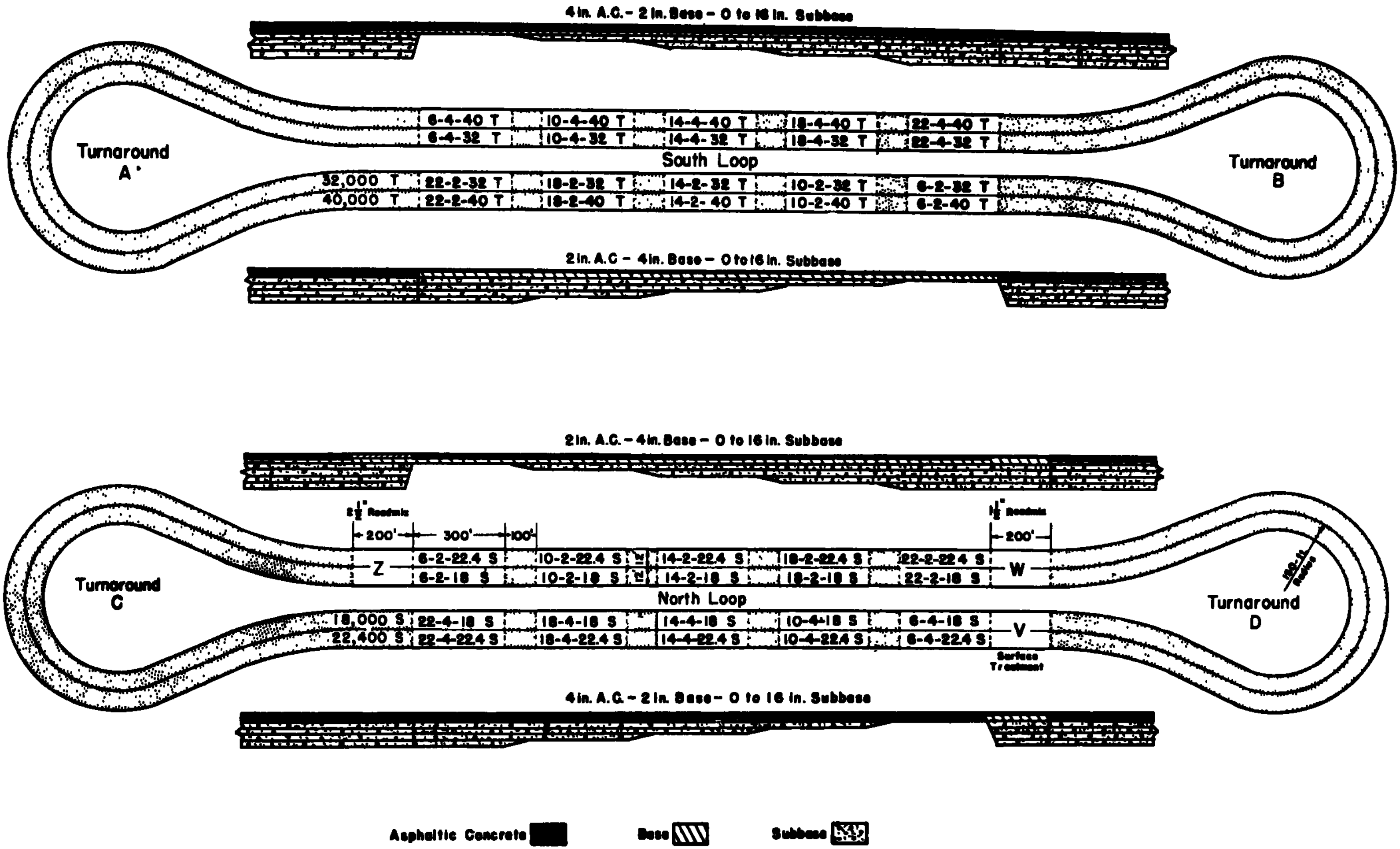


Figure 1. Schematic layout of test loops, WASHO Road Test.

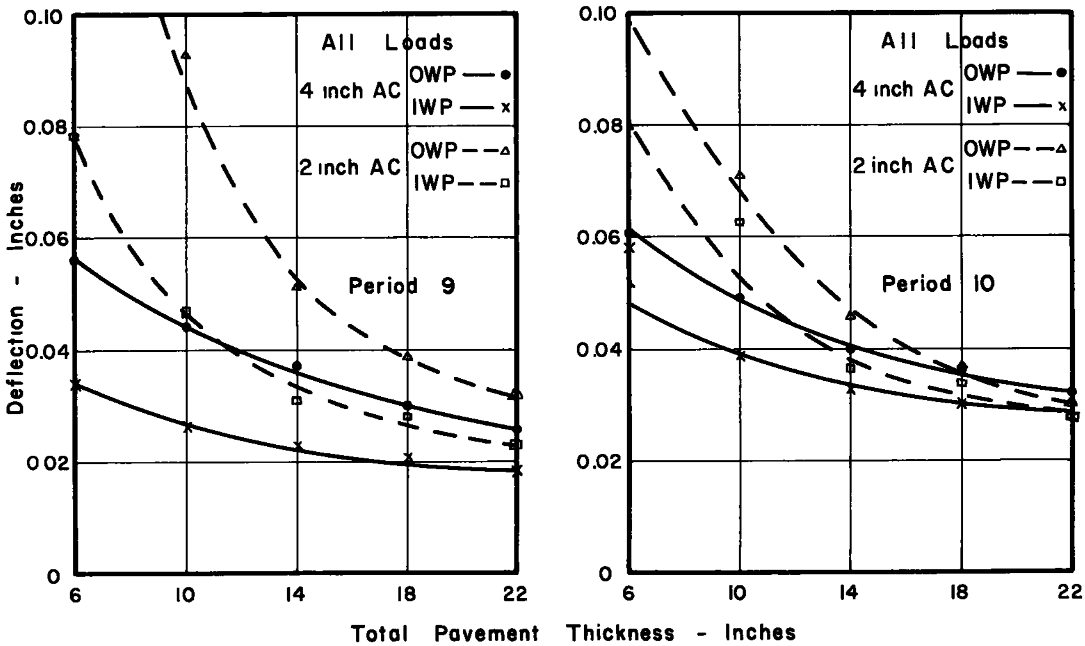


Figure 2. Pavement thickness-deflection relations.

Period	Date
3	June 10 - July 7, 1953
4	July 7 - July 24, 1953
5	July 24 - Nov. 21, 1953
6	Nov. 21 - Dec. 11, 1953
9	Feb. 23 - Apr. 7, 1954
10	Apr. 7 - May 29, 1954

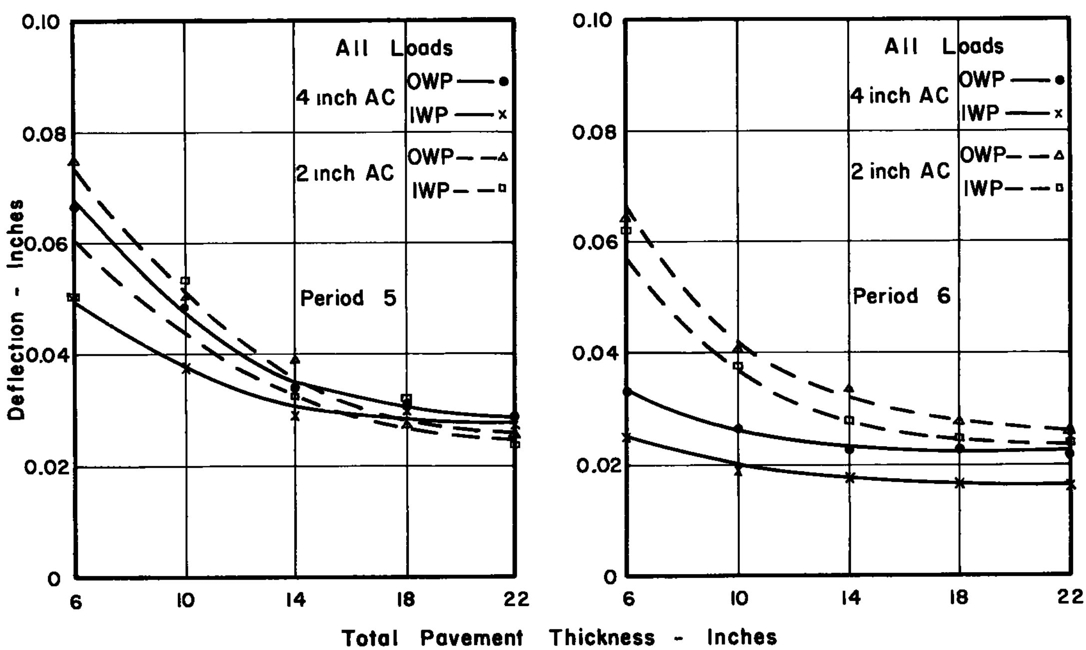


Figure 3. Pavement thickness-deflection relations.

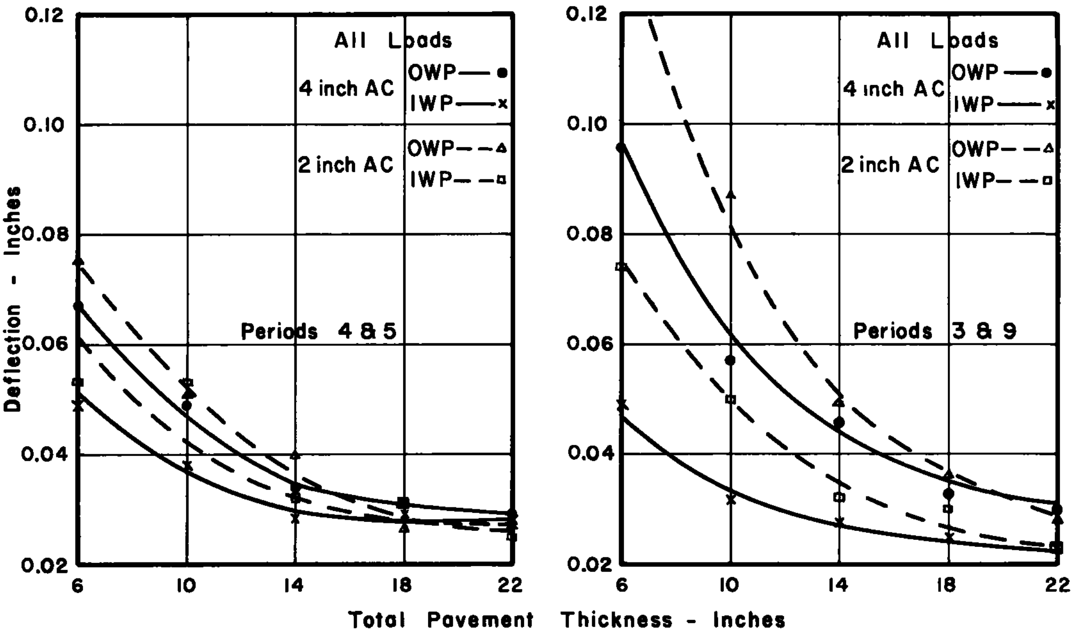


Figure 4. Pavement thickness-deflection relations.

SUPPLEMENTAL ANALYSIS OF THE ROAD TEST DEFLECTION DATA

In an effort to answer some of the questions raised by the WASHO test regarding the difference in behavior of the outer and inner wheel path areas of the pavement, a supplemental analysis of the deflection data was made. For example, it was attempted to determine from the analysis to what extent the thickness of the structure, the thickness of the bituminous surface, and the season of the year entered into the difference in behavior

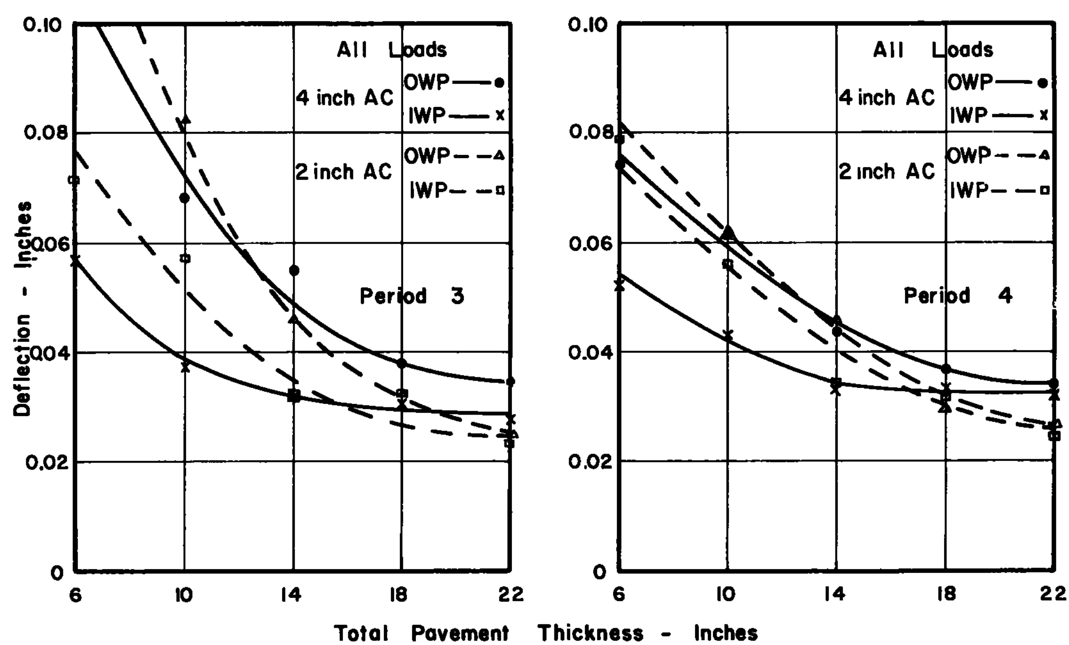


Figure 5. Pavement thickness-deflection relations.

TABLE 1

PAVEMENT DEFLECTION RATIO VALUES

		2 in. AC/4 in. AC						OWP/IWP				
	Wheel	Pavement Thickness					Thickness AC	Pavement Thickness				
Period	Path	22	18	14	10	6	Surface (in.)	22	18	14	10	6
3	Outer	0.77	0.87	0.94	1.11	1.25	2	1.08	1.18	1.31	1 54	1.77
	Inner	0.83	0.93	1.09	1.36	1.33	4	1.17	1 27	1.53	1.89	1.88
4	Outer	0.76	0.84	0.96	1.05	1.07	2	1.04	1 03	1.07	1.11	1.11
	Inner	0 76	0.91	1.20	1.37	1.33	4	1.03	1.12	1.35	1.40	1.42
5	Outer	0.90	0.91	1.03	1.06	1.09	2	1.04	1.03	1.09	1.16	1.23
	Inner	0.89	0.97	1.06	1.16	1.20	4	1.03	1.11	1.13	1.26	1.36
6	Outer	1.17	1.22	1.40	1.55	2.00	2	1.12	1.12	1.15	1.13	1 18
	Inner	1.41	1.47	1.55	1.85	2.24	4	1.35	1.35	1.28	1.35	1.32
9	Outer	1.23	1.30	1.47	2.02	2.46	2	1.39	1 44	1.60	1.94	1.80
	Inner	1.27	1.35	1.50	1.70	2 30	4	1.44	1.50	1 64	1.63	1.68
10	Outer	0.94	1 00	1.15	1.38	1 64	2	1 07	1.12	1.24	1.29	1.24
	Inner	1.00	1 06	1.15	1.36	1.65	4	1.14	1 20	1.24	1.25	1.25
3+9	Outer	0.97	1.06	1.16	1.31	1.41	2	1.20	1.37	1.50	1 64	1.80
	Inner	1.25	1 33	1.40	1.60	1.76	4	1.30	1.46	1.63	1.88	2.08
4+5	Outer	0.90	0.93	1.06	1 15	1 13	2	1.00	1.03	1.15	1.20	1.23
	Inner	0.93	1 00	1 06	1.13	1.17	4	1.03	1.10	1.16	1.24	1.30

Figures 2, 3, 4, and 5 show a series of pavement deflection-thickness relations for each of the foregoing periods and for each of two combined periods. These relations were developed by plotting the average of all the deflection tests made during the period on the pavement sections under all four test axle loads. In the majority of cases the values are the average of 150 or more individual tests, the average standard deviation of which varies with the magnitude of the deflection and is of the order of $\frac{1}{5}$ to $\frac{1}{3}$ of the mean.

As a general rule it appears that the relations are well supported by the test data. In several instances, however, the tests were made at locations where the pavement was partially distressed or was undergoing structural deterioration; in these cases some of the average values were erratic and considerable judgment had to be exercised in fitting the relations to the plotted points.

Table 1 lists values of the ratio of deflection of the 2- and 4-in. AC sections of pavement and the outer and inner wheel path portions of the pavement. These ratios were computed for the various thicknesses of pavement by using deflection values obtained from Figures 2, 3, 4, and 5.

Figure 6 is a plot of the outer and inner wheel path ratio values for each of the periods of traffic operation. From these data it is apparent that the values are greater generally for the thinner sections of pavement, particularly for major distress periods 3 and 9.

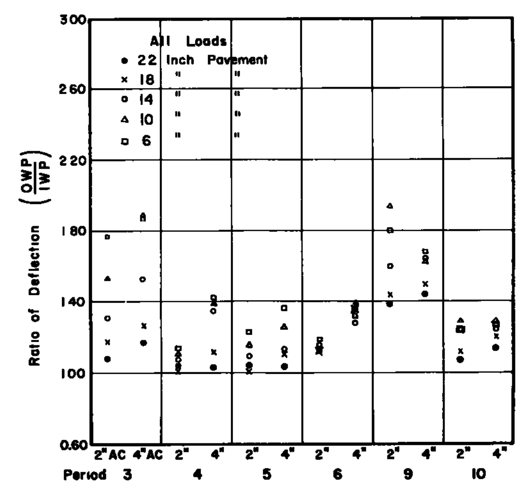


Figure 6. Pavement deflection-ratio values.

Figure 7 is a plot of the outer-inner wheel path ratio values as a function of the thickness of the pavement, both for periods 3 and 9 and periods 4 and 5 combined. According to the curves fitted to the plotted points there is a well-defined and consistent effect of thickness of the pavement structure (that is, the ratio values increase almost in linear fashion as the thickness decreases) and there is a pronounced difference in the values for the two designated periods. The fact that the ratio value is 1.00 for the 22-in. section in periods 4 and 5 suggests that the weakness of the

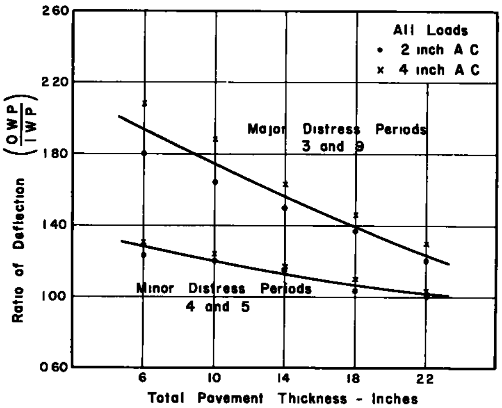


Figure 7. Deflection ratio relations.

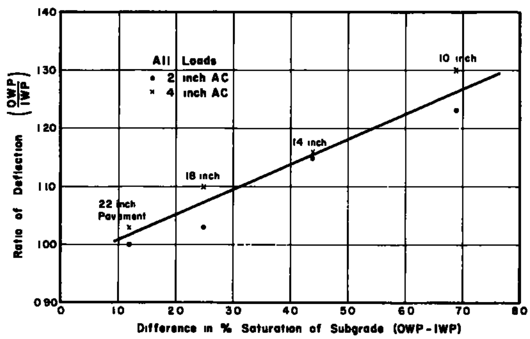


Figure 8. Effect of condition of subgrade soil on deflection of pavement.

edges of flexible pavements may under certain circumstances be overcome by increasing the thickness of the structure. Under other circumstances it may be greatly reduced by the use of increased thickness of pavement, but perhaps never eliminated.

CONDITION OF THE SUBGRADE SOIL

It is generally considered that the weakness of the edge of a flexible pavement is due to the more adverse condition of the supporting subgrade soil than may exist beneath the edge and/or to the discontinuity associated with the termination of the paved surface. Data concerning the condition of the subgrade soil of the test pavement during the summer months are given in Table 2. The values are the averages of determinations made beneath all the 22-, 18-, 14-, and 10-in. sections. The saturation values were computed from:

$$S = \frac{W}{1/d - 1/s} \qquad (1)$$

TABLE 2

CONDITION OF EMBANKMENT SUBGRADE SOIL DURING SUMMER MONTHS (0- to 12-in. Depth)

Pavement Thickness (in.)	Wheel Path	Moist. Cont. (%)	Dry Density (pcf)	Saturation (%)	Diff. in % Saturation
22	Outer	26.0	90.2	86.7	1.2
	Inner	25.3	90.7	85.5	
18	Outer	24.5	93.3	88.4	2.5
	Inner	24.3	92.5	85.9	
14	Outer	25.0	91.5	86.2	4.4
	Inner	23.3	92.2	81.8	
10	Outer	26.0	91.3	89.0	6.9
	Inner	24.3	90.7	82.1	

in which

W = moisture content (wt of water/wt. of soil);
d = dry density, in g per cc; and
s = specific gravity (2.55).

Figure 8 is a plot of the difference in the saturation values in the outer and inner wheel paths for each of the four thicknesses of pavement versus the ratio of deflection in the two wheel paths. The resulting curve serves to lend support to the supposition that the difference in deflection of the pavement in the outer and inner wheel paths during the summer months may have been due in large part to the difference in condition of the subgrade soil at the two locations. Why the deflection in the two wheel paths of the 22-in. section of pavement was about the same (ratio of 1.0) when there was a difference of 1.0 percent in the degree of saturation of the subgrade is not known.

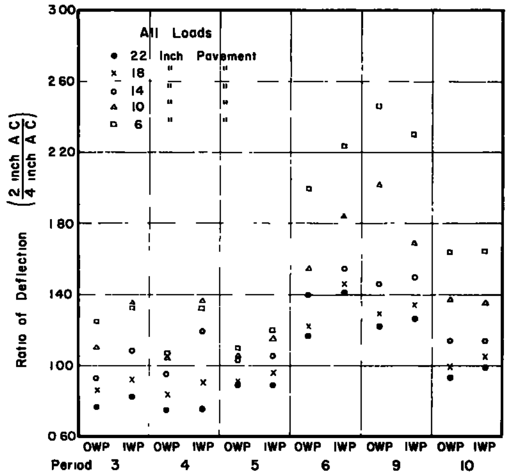


Figure 9. Pavement deflection-ratio values.

From the data available on the condition of the subgrade soil during the spring months it was not possible to explain why the outer-inner wheel path deflection ratios for the spring period were so much greater than during the summer months.

PAVEMENT DEFLECTION AND BEHAVIOR

Data concerning the ratio of deflection of the 2- and 4-in. AC sections of pavement are shown in Figures 9 and 10. According to Figure 9 there is little difference in the ratios for the warm weather periods (3, 4, 5, and 10). The ratios for cold weather periods (6 and 9), however, are significantly greater than for the other periods, indicating that the thicker AC surface sections offered considerably more resistance to deflection at low than at moderate to high temperatures.

In Figure 10 these ratio values are plotted as a function of the total thickness of the pavement. The marked difference in the indicated resistance of the 2- and 4-in. asphaltic concrete pavement sections in the cold and in the warm weather periods is clearly shown by the curves fitted to the plotted points. Also they show the pronounced effect of pavement thickness for cold weather period 9 and to a lesser degree its effect for warm weather periods 4 and 5.

These data imply that in cold weather areas where adverse subsurface conditions

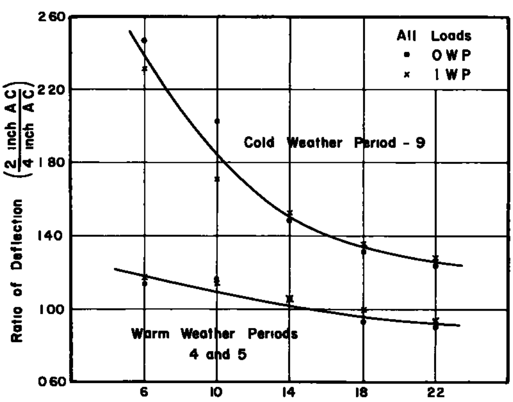


Figure 10. Deflection ratio relations.

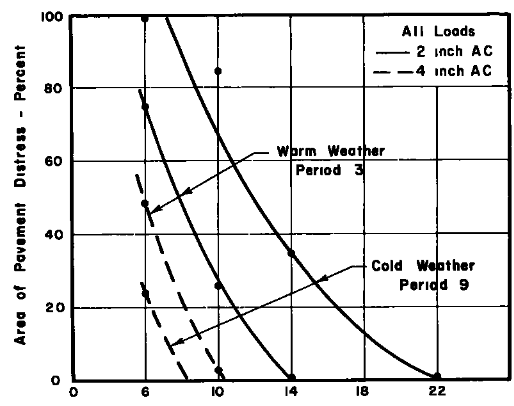


Figure 11. Pavement distress thickness relations.

due to frost action may reduce the ability of pavements to carry load, the use of relatively thick bituminous surfaces would be justified. Furthermore, that in warm weather areas the use of the thicker surface would be justified where there may be some question as to the ability of the basic pavement structure to support the prevailing or anticipated future traffic loadings.

Of significance in the foregoing connection are the data shown in Figure 11. Here the distress that developed in the 2- and 4- AC designs of pavement in the two periods in question is plotted as a function of the over-all thickness of the pavement. The resulting relations serve to demonstrate the marked difference in behavior of the two designs that occurred during the two critical periods of traffic operation.

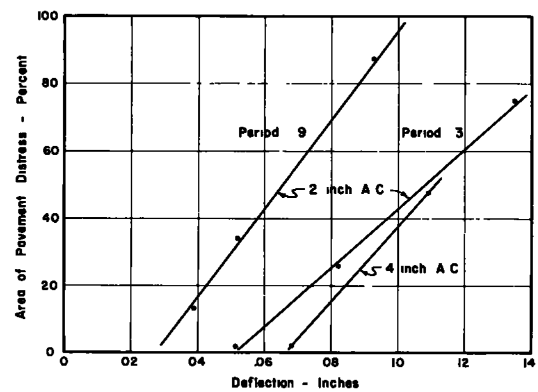


Figure 12. Pavement distress deflection relations—all loads.

As indicated in the statement of findings in the WASHO Test Road report, very little distress (551 sq ft, or 1. 6 percent of the total) occurred during period 5, when 45 percent of the total test loads were applied. This distress was about evenly distributed between the 2- and 4-in. AC sections, a development which serves to lend support to the relations shown in Figure 10 and the discussion pertaining thereto.

The report (p. 93) states:

"The traffic tests demonstrated the importance of the factor of thickness of the pavement structure. With very few exceptions, the thin sections underwent structural deterioration first and damage progressed in an orderly manner to the next increment of pavement thickness."

Figures 2, 3, 4, and 5 show that as a general rule the deflection of the pavement increased in a consistent and orderly manner as the thickness of the structure decreased.

This in itself would seem to indicate that there should be a reasonable degree of correlation between deflection, as measured in this test, and pavement behavior. Actually, this was indicated from the tests and observations made, and analysis of the data resulted in the selection of maximum values of deflection that were not considered to be associated with the development of structural distress in the pavement (that is, 0. 045 in. in warm weather and 0. 030 in. in cold weather).

The reexamination of the distress-deflection data resulted in the development of the curves shown in Figure 12. Here, as in Figure 11, distress is expressed as a percentage of the area of a particular section that was in good condition at the start of the period in question. The deflection values are the averages of all the tests made in the respective periods, the same values as those plotted in the previous graphs.

The curves drawn through the plotted points indicate that the distress varies as a linear function of the deflection. Although the three curves are practically parallel, the values for period 9 are significantly less than those for period 3. There are several possible explanations for this including the following:

1. The difference in the level of the temperature of the AC surface (period 3 being a warm and period 9 a cold weather period). As indicated previously, it was concluded in the initial analysis of the data that the pavement surface cracked or was distressed at lower deflections in cold weather than in warm weather.

2. Period 3 was the first extensive period of traffic operation and the pavement surface may have been more flexible than during period 9 after it had been subjected to almost uninterrupted traffic throughout the summer and fall months of 1953. During this time the density of the bituminous surface increased appreciably and this change

in the physical state of the material, together with a possible hardening of the asphalt cement, may have tended to reduce its flexibility.

SUMMARY

The information presented in this report serves to emphasize the importance of thickness of the pavement structure and the season of the year, both with regard to the difference in resistance to deflection of the edge and interior portions of the pavement and to the difference in resistance to deflection of the 2- and 4-in. AC surfaces.

During the major distress periods the data show that the deflection of the edges of the 6-, 14-, and 22-in. WASHO sections was 2. 0, 1. 6 and 1. 3 times that of the interior portions of the sections; during the minor distress periods these values are about 1. 3, 1. 2, and 1. 0. These values clearly indicate what might be accomplished in the way of balancing the load-carrying capacity of flexible pavements by increasing the thickness of the structure.

During the cold weather periods the deflection of the 2-in. AC surfaces of the 6-, 14-, and 22-in. sections was about 2. 40, 1. 50, and 1. 30 times that of the 4-in AC surfaces; during the warm weather periods these values are about 1. 2, 1. 0, and 0. 90. These values are considered an indication that the use of thick (4 in.) asphaltic concrete surfaces would be justified in areas where low temperatures are common. Also, that their use would be justified in warm weather areas where the basic pavement structure may be considered inadequate.

AASHO ROAD TEST STUDIES

One of the important decisions made during the planning stages of the AASHO Road Test was to construct a series of representative sections of flexible and rigid pavement to be used only for special observations and measurements. These sections form the smallest of the project's six test loops. Its tangents are 2,100 ft long, with flexible sections on one side and rigid sections on the other. In the 32 flexible sections, surface thicknesses include 1, 3, and 5 in. of AC; base thicknesses, 0 and 6 in. of crushed stone; subbase thicknesses, 0, 8 and 16 in. of sand-gravel. Eight of the flexible sections will be used for bi-weekly subsurface observations and tests and 24 for deflection measurements. These measurements will cover a wide range in temperature conditions of the AC surfaces and a variety of conditions of the subsurface components.

From this work it is anticipated that a great deal of additional information of the same general nature as that presented in this report will be obtained regarding the ability of flexible pavements of different thicknesses of surface, base, and subbase to support load in both the outer and inner wheel paths under different climatic conditions.

Discussion

W. H. CAMPEN, Manager, Omaha Testing Laboratories, Omaha, Nebraska — It may be that the larger deflection at the edge of the pavement is due to lack of lateral support. If this is the case, the lateral support could be provided to correct the situation, rather than increasing the edge thickness.

It is the writer's understanding that during the testing period at the WASHO project lateral support was provided on certain portions. Perhaps the author knows what was done and what results were obtained.

The writer happens to be consulting engineer for Douglas County, Nebraska. This county has had an extensive program for about 9 years and has constructed about 120 miles of pavement consisting of 4 to 6 in. of stabilized base plus a prime and double armor coat. For lateral support a 4-in. by 12-in. concrete header or curb has been used. The top of the curb is flush with the top of the base. This type of construction has eliminated distress along the edges and has prevented the usual progressive base disintegration from the edge toward the center. The roads are on the secondary road system and 20 ft in width. The curb costs about 17 percent of the entire cost of construction.

CLOSURE, A. C. Benkelman — Shortly after test traffic was started on the WASHO pavement, the shoulders of part of three of the test sections were paved. This proved highly effective in reducing the deflections and the rate of development of distress of the original pavement edge. However, it was not determined how much of this reduction was due to the additional lateral support afforded by the shoulder paving or how much was due to an improvement of the condition of the subsurface components underlying the shoulder.

The AASHO Road Test has been planned in such a way that a great deal of factual information should be obtained on the question raised by Mr. Campen.

Flexible Pavement Design in Idaho

L. F. ERICKSON, Materials Engineer, Idaho Department of Highways

This paper reviews flexible pavement design methods used by the Idaho Department of Highways since soils tests were begun in 1938. California Bearing Ratio tests were used prior to World War II, but in 1942 an empirical soil formula similar to group index was adopted and used until 1950 for evaluating subgrade soils for flexible pavement design.

The Hveem stabilometer has been used for bituminous work since 1938, but construction of a kneading compactor in 1950 materially expedited the work. The paper by Hveem and Carmany (1) on factors underlying the rational design of pavements in 1948 led to adoption of resistance value criteria for the design of flexible pavements.

The paper reviews other criteria considered by Idaho in the design of flexible pavements. It concludes that after seven years use of resistance value criteria, this method is satisfactory and proper utilization of these data will do much toward reducing the number of subgrade, base, and surfacing failures on Idaho roads.

THE FIRST ROUTINE use of soil testing by the Idaho Department of Highways was started in 1938 with emphasis on compaction control. Previous use of soil test data had been limited to only a few isolated locations where it was evident that unusual conditions existed and further knowledge was desired. A soil survey and soil profile was made for each project following essentially the procedures set forth in AASHO T 86, "Surveying and Sampling Soils for Highway Purposes," and the soil was classified in accordance with the Bureau of Public Roads classifications. All test information was reported to design and construction engineers during this period and they became fairly familiar with the soil test data and noted some relationships between soil types and field performance.

During 1940 the laboratory began to evaluate the soil data and to recommend a total thickness of base and surfacing for use in design. At this time the evaluation was based almost entirely on the judgment of the evaluator and was seldom supported by information from field performance. Nevertheless, these evaluations did much toward making designers and construction personnel conscious of soil types and characteristics.

In general, the thicknesses proposed by the laboratory were accepted because the recommendations were for thicker base and subbase courses than had been provided previously. Some engineers, being familiar with particular local soils and conditions, were better able to correlate performance with soil types locally and in some instances increased the thickness above that recommended by the laboratory. A soil survey using essentially the methods outlined in AASHO T 86 is still in use today, and electrical resistivity equipment has been used to good advantage since 1956. In 1957, seismic equipment was purchased as a further aid in this work. All essential test data are shown on the soil profile, together with recommendations for total thickness of the pavement structure and any needed special treatment of subgrade.

Prior to 1942 the California Bearing Ratio test was made on nearly all soil samples, compacting the sample under a static load of 2,000 psi. Although it was realized that this loading did not give a density comparable to that obtained in the test for moisture-density relationship, it was believed that, after the 4-day soaking period, practically the same results would be obtained for CBR as those to be expected had comparable densities been obtained. About 1940, tests such as the centrifuge moisture equivalent, shrinkage limit, and hydrometer analysis were discontinued as routine tests. A linear shrinkage test was adopted in lieu of the shrinkage limit, as it was a more expedient test and measures equivalent properties.

EMPIRICAL SOIL NUMBER

North Dakota, in 1942, published a formula in which a soil number was computed utilizing the soil test data used to determine the BPR soil classification. The soil number thus obtained was reportedly correlated with service behavior. But there were many different opinions regarding the thickness of pavement structure to be provided. Also, to add confusion, there were inconsistencies in the evaluations from each individual evaluator and these inconsistencies proved to be embarrassing to the laboratory.

The empirical soil number offered a solution to this problem, as well as a means toward obtaining better correlation with field performance. The North Dakota formula was revised, using only those tests that were routine in Idaho. The Idaho soil number was correlated with previous evaluations, which were confirmed by road performance in local areas. The formula used in computing the soil number in Idaho has not been changed since its inception and is as follows:

$$\text{Ballast Number} = A + B + C + (D + E + F + G) + H \qquad (1)$$

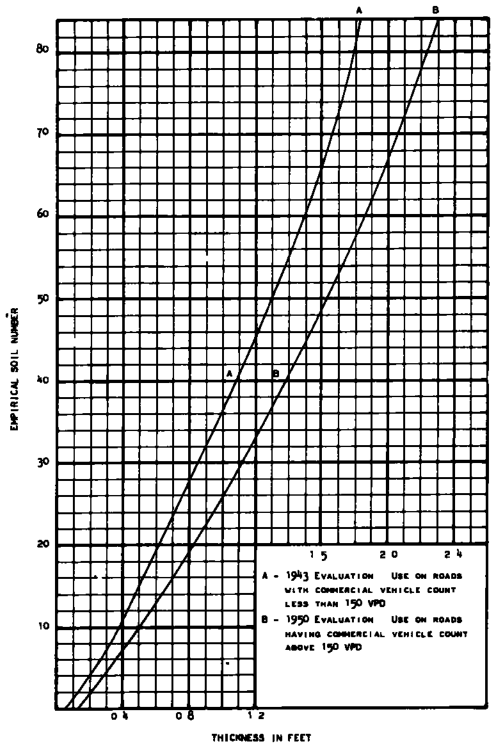


Figure 1. Design pavement thickness chart based on empirical soil number.

in which

$$A = \frac{\%\text{ pass No. 10} - 50}{10} \quad (\text{if less than 50 percent pass, } A = 0);$$

$$B = \frac{\%\text{ pass No. 40} - 30}{10} \quad (\text{if less than 30 percent pass, } B = 0);$$

$$C = \frac{\%\text{ pass No. 40}}{40} \quad (\text{if more than 40 percent pass, } C = 1);$$

$$D = \frac{\text{Liquid limit} - 15}{3} \quad (\text{if liquid limit less than 15, } D = 0);$$

$$E = \text{Plasticity index} - 5 \quad (\text{if plasticity index less than 5, } E = 0);$$

$$F = \frac{\text{Rose moisture equiv.} - 15}{10};$$

$$G = \text{Lineal shrinkage (use nearest whole number); and}$$

$$H = \frac{130 - \text{weight/ cu ft}}{3} \quad (\text{use maximum dry weight from moisture-density test}).$$

The factor C is used to reduce the numerical value resulting from adverse soils constants in granular materials in proportion to the amount of material passing the No. 40 sieve. This is because the more granular material there is in a sample the more stable the material should be regardless of adverse soil characteristics when considered as a subgrade material.

The application of this formula and the associated design curve was changed in 1950 to give about 25 percent increased total thickness of pavement structure. The greater thickness provided by this revision was based on results of an investigation of the behavior of bases and pavements placed just prior to and immediately after World War II. Both the original (1943) design curve and the 1950 curve are shown in Figure 1. It was recognized that the amount and type of traffic had a significant part in pavement performance, so a total commercial vehicle traffic count of 150 ADT was selected as the dividing point for design purposes.

There is a similarity between the Idaho empirical soil number and the group index, although they give only a general correlation. A comparison made several years ago indicated that a better correlation resulted when the weight per cubic foot and linear shrinkage values were omitted from the Idaho formula. These values are believed, however, to indicate properties that are significant in performance and their use has been continued in the computation of the Idaho soil number.

ADOPTION OF RESISTANCE VALUE CRITERIA

The California Bearing Ratio test was discontinued after World War II, as it was then realized that static compaction was unsatisfactory. Impact compaction, although it improved the test, required more time than permissible due to personnel limitations, if several specimens were prepared. Design charts proposing pavement thicknesses from the CBR test data appeared to give excessive thicknesses in the opinion of some of the engineers and this also discouraged its use.

Idaho had been using the Hveem stabilometer for testing bituminous surfacing for several years and in 1950 constructed a pneumatically-driven kneading compactor to be used in preparing specimens of bituminous mixtures for testing. It was soon learned that the compactor gave entirely different results for stability than did static compaction. This information was checked by testing unstable mixtures taken from existing pavements and it was determined that kneading compaction often gave stability values considerably below those obtained for static compaction on the same mixtures. Since 1950 there have been numerous occasions for testing stable and unstable bituminous mixtures from existing pavements and the kneading compactor has permitted the stabilometer to indicate the difference readily, whereas static compaction has rarely given the same indication. The criteria in use by the California Highway Department

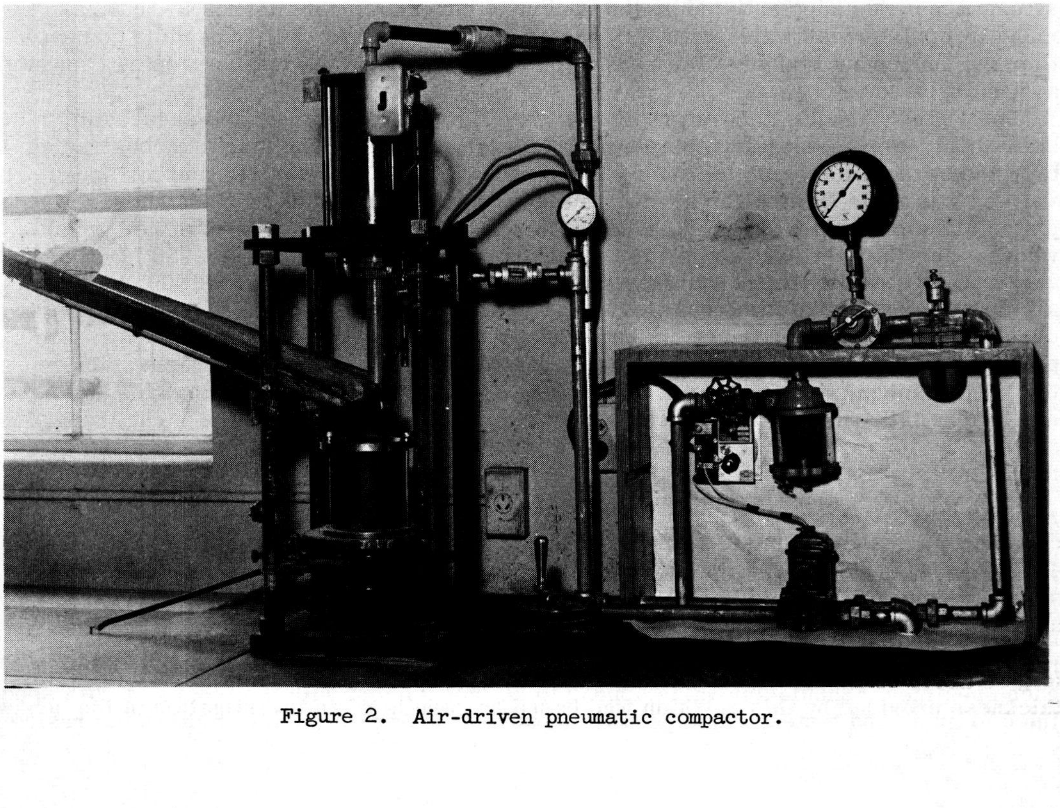


Figure 2. Air-driven pneumatic compactor.

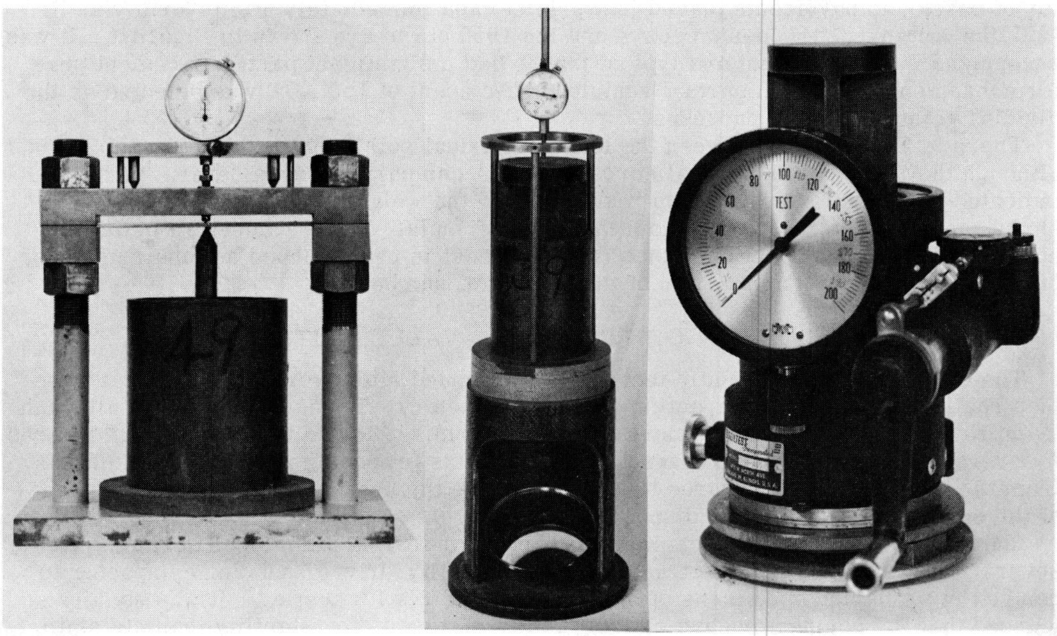


Figure 3. Expansion pressure frame (left), Washington visual saturation indicator (center), and Hveem stabilometer (right).

for bituminous mixtures were verified in every case investigated. The design procedure outlined by Hveem and Carmany developed considerable interest, and because of the information obtained from investigating unstable bituminous surfacings it was decided that the stabilometer method should be tried for soil testing. The need for a strength test was substantiated by the review of pavement performance in 1950, as many inconsistencies were noted. It was concluded, however, that kneading compaction and the Hveem stabilometer give reliable results when the test is properly conducted and interpreted.

The Forest Service Laboratory at Arcadia, Calif. had designed and successfully built an air-driven kneading compactor that was practical. The machine constructed by Idaho, except for electric timer and controls, air pressure valve, and air speed valves, was entirely manufactured in the Idaho highway shops using the double-acting piston principle of the Forest Service machine. Air was used to operate the turntable, whereas the Forest Service operated its turntable electrically. Since 1950 Idaho has constructed three of these machines. The latest (1957) cost less than $800, including all controls, with shop time charged at the rate of $4.50 per hour. Figure 2 shows one of these compactors set up in the soil laboratory. Figure 3 shows the expansion pressure frame, the Washington visual saturation indicator, and the Hveem stabilometer. Figure 4 is a trace of the foot pressure developed while compacting soil in a normal manner; that is, at 30 strokes per minute at 250-psi foot pressure. The trace, obtained by means of SR-4 strain gage equipment, indicates that no impact is imparted to the soil specimen.

Preparation of test specimens is expedited by this method of test. Two men are able to prepare four soil specimens at varying moisture contents for each of the 25 or more samples tested each week. One additional man working half-time can increase this output to nearly 40 samples a week. These men weigh out the material for each specimen, moisten the sample and permit it to condition overnight, compact the specimens, determine saturation pressure, measure and weight specimens for density, measure expansion pressure, and compute the resistance value and moisture content of each specimen. Should any value appear to be inconsistent with the others, a fifth specimen is made and tested. This permits construction of a satisfactory curve for resistance value versus expansion pressure and resistance value versus saturation pressure. Bituminous mixtures are tested in another section of the laboratory; if the need should ever exist, a comparable output could be attained in this section.

DETERMINATION OF RESISTANCE VALUE, EXPANSION PRESSURE, AND SATURATION PRESSURE

The procedure used for determining the resistance value, saturation pressure, and expansion pressure follows the procedure developed by California, except that a compaction pressure of 250 psi is used. This is the unit pressure specified for sheepsfoot rollers in the 1950 Idaho specifications. This value was chosen because it was believed that greater pressures would result in too high densities and thereby possibly produce higher resistance values than should be depended on in relationship to the densities specified or attained in construction.

Each specimen is made using material from which material coarser than $^3/_4$ in. is removed and without replacing this oversize with an equal amount of No. 4 to $^3/_4$-in. material as is advocated in some other tests. Sufficient material is prepared and moistened to make at least five test specimens of the proper height and weight after compaction. This material is permitted to condition over night so that all of it has been permitted to absorb moisture. The moisture content of the material is then determined and sufficient material is weighed out for each specimen.

The first specimen is compacted at the existing moisture content, with the compactor applying a foot pressure of 250 psi for 140 strokes. The specimen is then placed on the Washington visual saturation indicator and compressed by static loading until five of the six segments appear wet. The load applied is recorded as the exudation or saturation pressure. The weight, height, and weight per cubic foot of the specimen are then determined. The specimen is then set aside and permitted to rebound.

A second sample is then moistened, adding approximately one percent water as indicated by experience, and the foregoing procedure is repeated. All specimens are permitted to rebound for approximately one hour. They are then placed in the expansion pressure frame and the stand adjusted to apply a surcharge load of about 0.1 psi to seat the stand. The mold is filled with water to about $1^1/_2$ in. above the surface of the soil and the pressure developed by the soil in endeavoring to expand is determined after 16 to 18 hr by the deflection measurement of the calibrated bar. The specimen is then pressed from the mold into the Hveem stabilometer and the resistance value is determined by applying a vertical load at a strain rate of 0.05 in. per minute and reading lateral pressures for vertical loads of 500, 1,000, 1,500 and 2,000 lb. The vertical load is then reduced to 1,000 lb and the lateral pressure is reduced to 5 psi by means

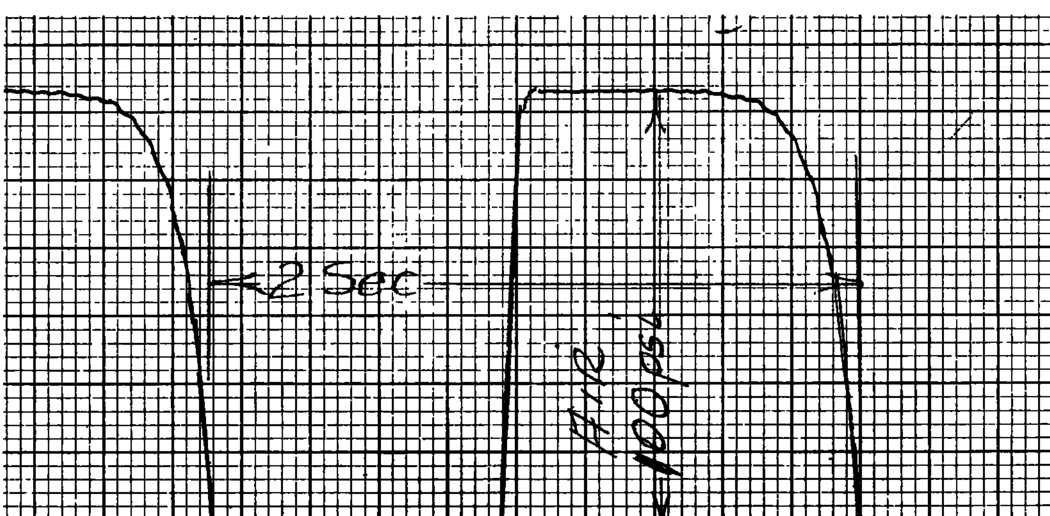


Figure 4. Foot pressure diagram for compactor.

of the displacement pump, permitting the vertical load to continue to drop. The displacement pump turns necessary to increase the lateral pressure from 5 psi to 100 psi are noted and recorded as the displacement. A moisture determination is made of the material so that the amount absorbed during the expansion pressure test and the moisture content at the time of the R-value test will be known. The resistance value (R value) is calculated from:

$$R = 100 - \frac{100}{\frac{2.5}{D}(P_v P_h - 1) + 1} \tag{2}$$

in which

D = Displacement turns;

P_v = Vertical pressure (160 psi at 2,000 lb); and

P_h = Horizontal or lateral gage pressure for 2,000-lb load.

Resistance values for each specimen are plotted against the corresponding saturation pressures and a smooth curve is drawn through the points. The R value for a saturation pressure of 5,000 lb is taken as the R value of the material, although a greater or lesser saturation pressure could be used. This is the value used by the California Department of Highways. Similarly, the expansion pressure is plotted against R values and the expansion pressure noted where R value and expansion pressure give equal total pavement thickness for an assumed traffic index of 7 and a 130-pcf unit weight of material. These curves are reproduced on the laboratory report (Fig. 5) submitted to designers and field personnel.

Select borrow, select base, and base materials are tested for R value and expansion pressure as previously outlined whenever the material is plastic or has properties that may be considered doubtful for quality, such as material subject to degradation. Such material will degrade in the compactor; therefore, 1,000 strokes at 250 psi have been applied to deliberately degrade the material before testing it for R value. Although this does not represent the end product of degradation in the field, it may nevertheless indicate how seriously the R value is reduced by degradation.

EVALUATING TRAFFIC

A traffic index is computed for each project, using data obtained from reports of the planning survey. The estimated traffic data projected for 20 years hence are presented in a design brochure prepared by the design engineer. This brochure includes essential information for the facility to be provided, including geometric sections of the highway and locations of frontage roads, interchanges, and separations. Until 1957 the traffic index was computed by the formula presented by Hveem and Carmany (1). A more recent formula has been developed (it is understood that is is to be included in a proposed manual of the AASHO Committee on Design), as follows:

$$TI = 1.35(EWL)^{0.11} \tag{3}$$

in which EWL = 5,000-lb equivalent wheel loads.

Idaho traffic data have been reviewed twice since 1950 and it is noted that the average axle loadings have increased from slightly less than 12,000 lb in 1950 to about 14,500 lb in 1956. This increase results in a large increase in the EWL value for the same number of vehicles. Using available traffic data, the traffic index has been computed for varying numbers of heavy vehicles; that is, vehicles having axle loadings of more than 10,000 lb. Figure 6 gives the traffic index to be used for these various groupings of vehicles and a design chart plotting total thickness against R value for varying traffic index numbers assuming a cohesiometer value of 100 for granular material. The total depth of pavement structure to be provided is computed from:

$$T = \frac{0.095\ TI\ (90 - R)}{\sqrt[5]{C}} \tag{4}$$

in which

T = Total thickness, in inches;
TI = Traffic index;
R = R value; and
C = Cohesiometer value assigned various surfacing materials, i.e., P.C. concrete, plantmix, roadmix, cement treated base, etc.

The traffic index is varied from 3 to 11.5 This provides for roads carrying only a couple of heavy vehicles per week to a highway that would carry 2,900 or more per day.

Copies to: Highway Engr. District Engr. Resident Engr. B.P.R. File

STATE OF IDAHO
DEPARTMENT OF HIGHWAYS
Materials Laboratory
BOISE

IH-803-11-57

Lab. No. 125855

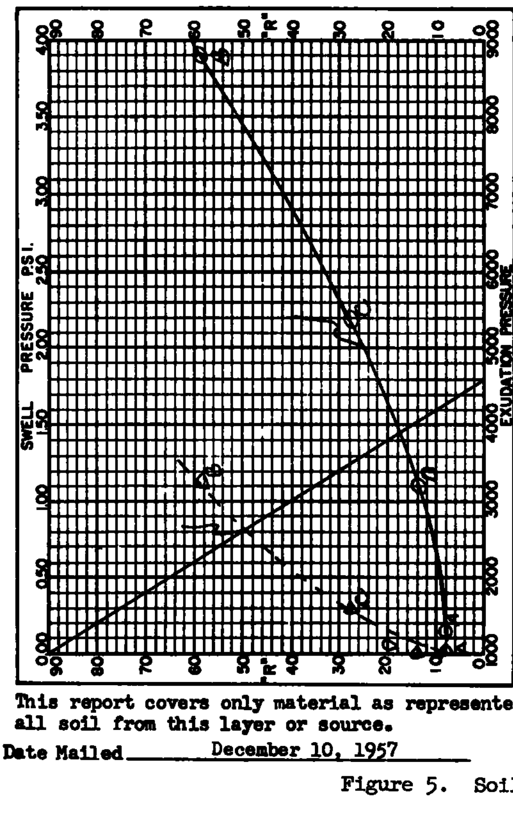
Report of Tests on SOIL Embankment & Subgrade

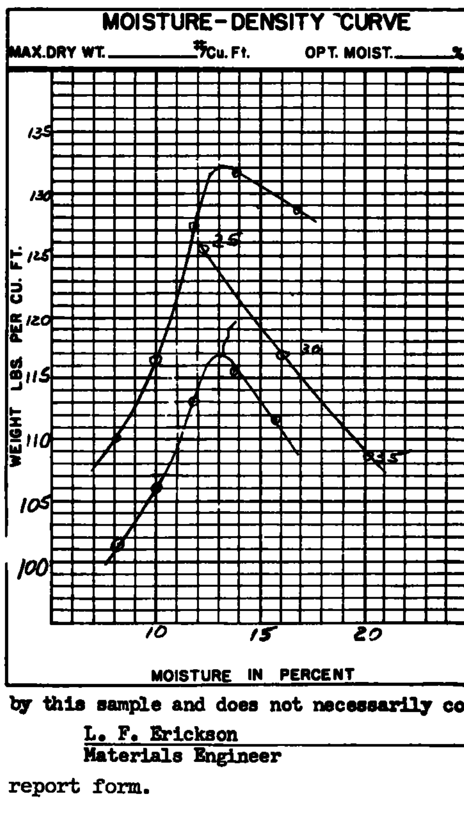
Project I-IG-1032(5) Blackfoot-Bonneville Co.Line County Bingham
Submitted by J. Alford Ident. No. JA/4549-2213/8-P
Layer No. 4 Source No. Depth,feet 0.6'-3.0' Station 1135, Rt. Lane
Description of Soil Silt and Gravel Date Sampled 11-27-57 Received 12-2-57

Mechanical Analysis % Passing				
3" Sq.				
2" Sq.	100			
1" Sq.	92			
3/4" Sq.	87			
1/2" Sq.	81			
No. 4	76			
No. 10	76			
No. 20	74			
No. 30	74			
No. 40	72			
No. 50	71			
No. 100	61			
No. 200	52			

Soil Constants

Liquid Limit 22 "R" Value 25
Plastic Limit 18 Swell Pressure Psi 0.82
Plasticity Index 4 Eq. "A" No. 18
Field Moist. Equiv. 20.6 Texture Class'n Silt Loam
Linear Shrinkage 4.0 Soil Class'n A-4(3)
Specific Gravity 2.68
Remarks





This report covers only material as represented by this sample and does not necessarily cover all soil from this layer or source.

Date Mailed December 10, 1957

L. F. Erickson
Materials Engineer

Figure 5. Soil report form.

TRAFFIC INDEX	HEAVY VEHICLE A.D.T.	
3.0	2 OR LESS/WEEK	RESIDENTIAL STREETS.
4.0	1 OR LESS/DAY	FRONTAGE ROADS WITH NO TRUCKING.
5.0	1-4	FRONTAGE ROADS NOT SERVING WAREHOUSE OR INDUSTRIAL TRAFFIC.
5.5	4-12	COUNTY ROADS OF VERY LIGHT TRAFFIC VOLUME.
6.0	12-20	NORMAL HIGHWAY TRAFFIC INCLUDING PRIMARY COUNTY ROADS WITH NORMAL TYPE AND DISTRIBUTION OF HEAVY VEHICLES.
6.5	20-50	
7.0	50-80	
7.5	80-140	FRONTAGE ROADS SERVING WAREHOUSE AND INDUSTRIAL TRAFFIC.
8.0	140-275	
8.5	275-550	
9.0	550-850	
9.5	850-1350	
10.0	1350-1750	
10.5	1750-2350	VERY HEAVY COMMERCIAL TRAFFIC. INDUSTRIAL TRAFFIC SUCH AS ORE AND LOG TRUCKS OF ABNORMAL NUMBERS WITH SUPPORTING DATA FURNISHED.
11.0	2350-2900	
11.5	2900 & OVER	

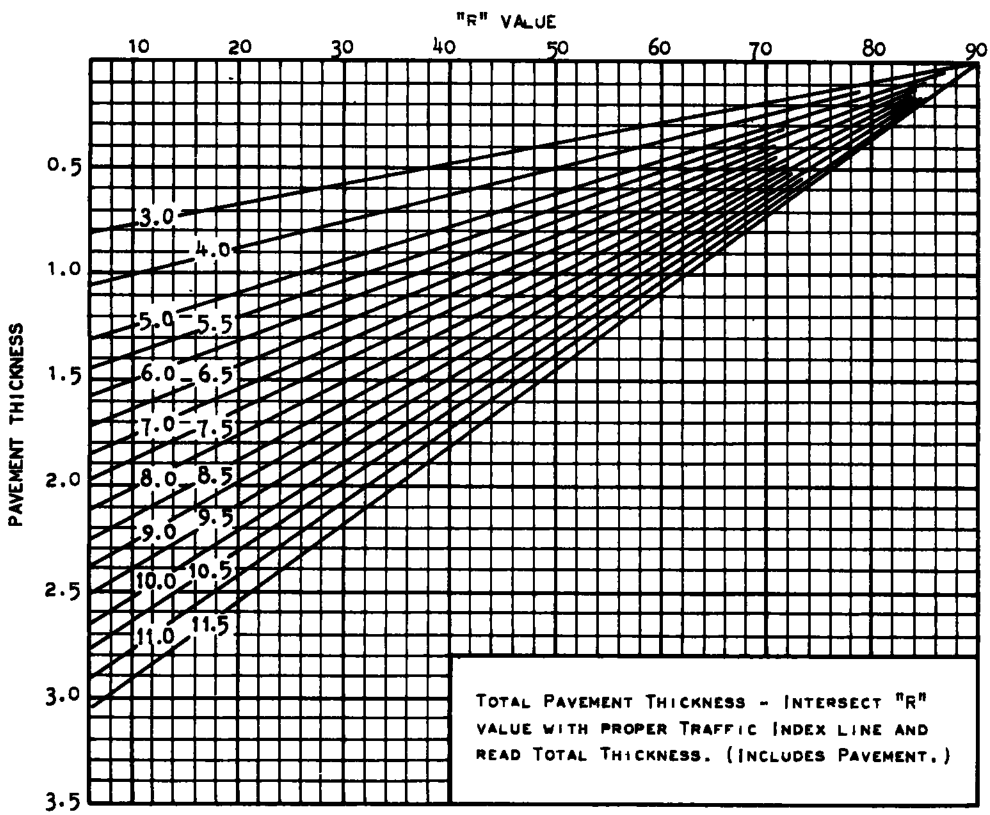


Figure 6. Design pavement thickness chart—traffic index and resistance value.

The normal variance of the traffic index used in Idaho ranges from 6. 0 to 10. 0. Values less than 6 are used for frontage roads not serving heavy vehicles and for some county roads of extremely light traffic volumes. A traffic index of 3 is used where only passenger car and pickup traffic use the roadway or street. The state has a few roads where most of the traffic involves heavy vehicles, such as logging trucks or ore trucks, which are all loaded to maximum legal axle capacity. In one case, approximately 600 ore trucks daily, each 5-axle and carrying a gross load of 72,000 lb travel one side of a secondary highway for a distance of about 12 miles from late spring when load limit restrictions are lifted until the mine is shut down in the winter. Highways carrying this type of traffic must be given special consideration, bearing in mind the period of the year when the highway is being used and providing a design adequate for that period. Idaho has many miles of county highways that serve only limited traffic; some have a traffic count of less than 50 vehicles per day. Many of these roads are located in remote areas and although a need does exist for a serviceable road, available funds permit only a minimum facility. Due to the extremely low traffic volume, many of these roads have been designed using a traffic index of 5 or 5. 5 and utilizing selected local material to avoid crushing operations or at least to keep such processing to a minimum.

Reductions in total thickness of pavement structure due to the use of cohesiometer values for cement-treated base or plantmix have rarely been utilized. This concept is new to Idaho and although this method of design probably will be adopted, traffic increases in recent years have so far exceeded expectations that there is an inclination to forego this reduction in thickness as an additional safety factor.

OTHER FACTORS CONSIDERED

Several other factors are considered in the design of flexible pavements. Fine-grained plastic soils used in the subgrade are covered with a layer of dense-graded aggregate or sand to prevent intrusion of the soil into the interstices of coarser base material. Normally this blanket course is about 3 in. thick. For fifteen years, Idaho has used as a guide criteria that any soil possession more than 70 percent passing the No. 200 sieve and having a P. I. greater than five and a linear shrinkage greater than five, should be blanketed with a dense-graded aggregate or sand. These criteria may be too conservative, but to date it has not been possible to determine better standards. Generally, where sand and gravel are available a blanket material is not required, because the soil types in areas containing sand and gravel deposits are generally sandy in nature and rarely very plastic. Perhaps of greater importance, is the fact that the sand-gravel deposits generally contain sufficient materials passing the No. 10 and No. 40 sieves to serve adequately as a blanket material. These blanket course materials are required to have 25 percent passing the No. 10 sieve if ¾-in. maximum and 20 percent passing the No. 10 sieve if 2-in. maximum. This requirement makes the production of stone aggregates costly when this is the only material available for use.

The Idaho standard specifications require that unsuitable material at grade points be excavated and replaced with approved material, preferably granular in nature. But there are a variety of opinions as to what constitutes "unsuitable" material. Therefore, included on the soil profile are notes and limitations of the areas to be excavated, giving depth of excavation below finished grade. These areas are also shown on the construction plans and whenever practicable the plans provide that the excavated area be sloped for natural drainage. Close attention to this detail is worthwhile and avoids subgrade trouble in critical areas where failures have so often occurred. Figure 7 is an artist's sketch illustrating the method of excavating and backfilling.

Where frost is not a problem, a select soil may be used to backfill grade points deemed necessary to be excavated. Where frost troubles are common, a granular material is required even though base must be produced for this purpose. Special attention is given to all drainage. Depths below subgrade which are commonly excavated and backfilled vary from 0. 5 ft where frost is no problem to more than 1 ft where frost boils are common. This practice is not perfect in its application as frost boils do occur at other locations than grade points.

The practice of excavating grade points is controversial and has been subject to many

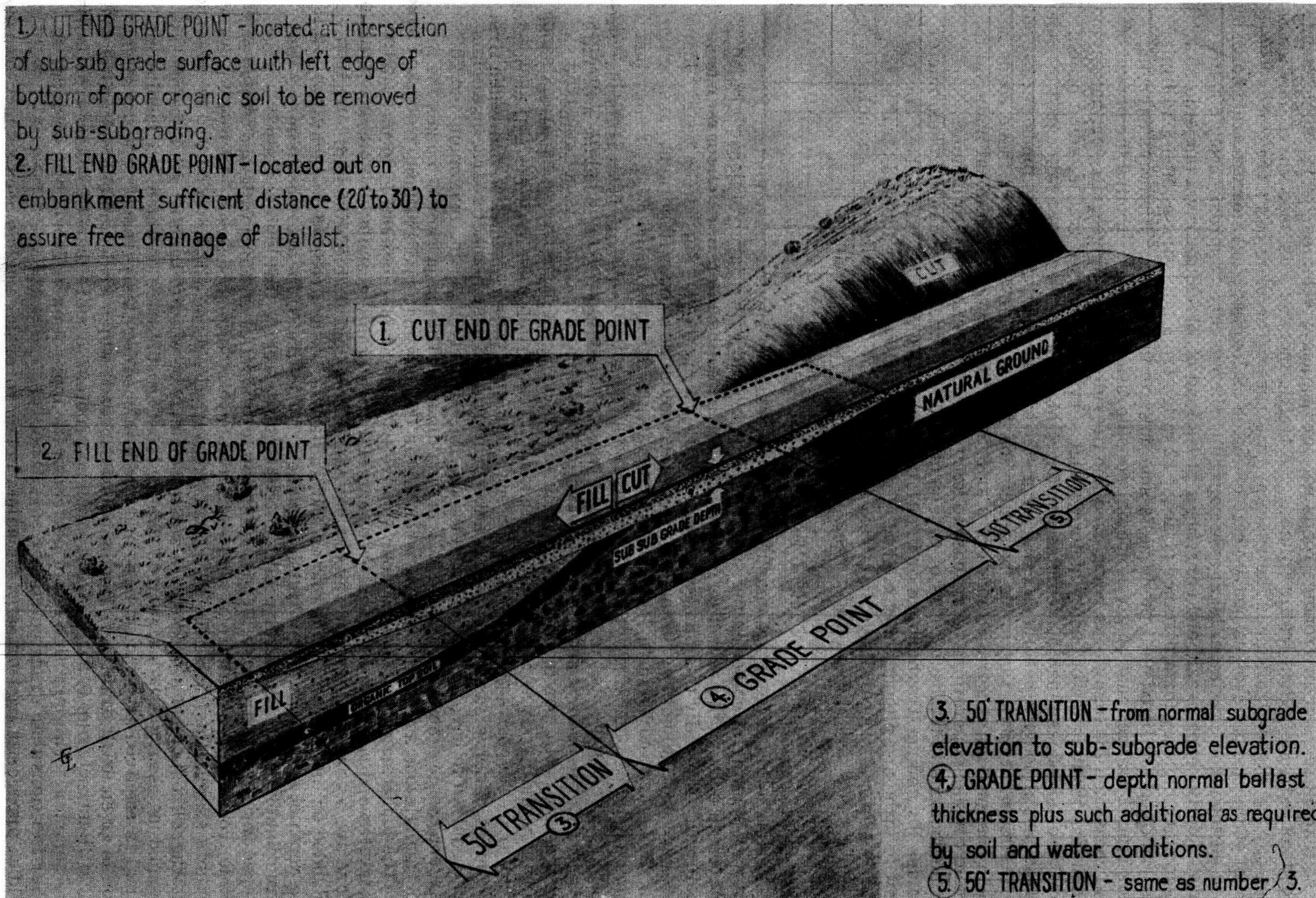


Figure 7. Method used to reinforce grade points.

Date

Project No.: I-IF-1032(5)
Section Blackfoot-Bonneville Co. Line
County Bingham

STATE OF IDAHO
DEPARTMENT OF HIGHWAYS
Materials Laboratory
BOISE

Sheet 1 of 1
Projected Traffic Data
Total V/Day 10640 1975
Commercial (Does not include panels & pickups) 1120

Curve "B" Traffic Index 9.5

Station	Lab.No.	Layer No.	Depth	Class'n	L.L.	P.I.	FME	L.S.	% Passing #10	% Passing #40	% Passing #200	Moist-Dens. W/C.Ft	Moist-Dens. Opt. Mois.	Eq. "A"	"R" Value	Expansion Pressure psi	Recomm. Thick.	Blanket
1135 Rt.	125855	4	0.6'-3.0'	A-4 (3)	22	4	21	4.0	76	72	52	116.8	13.0	18	25	0.82	2.0	No

GENERAL REMARKS: This evaluation is based upon the assumption that the soils evaluated will be used in the top subgrade layers and that standard drainage and compaction will be provided. The above recommendations for total thickness are considered for average climatic conditions, traffic, availability of base and surfacing materials, etc. One amply designed section over a group of soils is considered good practice for the sake of uniformity of section thickness.

Figure 8. Soil evaluation form.

extremes. Designers in a district having very little difficulty with frost boils may provide better than 1 ft of subgrade excavation, whereas other districts having a definite problem may ignore or only partially accept the criteria because some engineers believe the boils are impractical of correction. Progress is being made, however, and with these areas shown on the plans a more effective treatment probably will be obtained. The subgrade treatments designed recently are much more reasonable than when first proposed.

STANDARD DESIGNS

Idaho has endeavored to standardize the thicknesses of bituminous surfaces and base courses as much as possible. The interstate highways are designed to have a future 0.1-ft course of plantmix with 0.3 ft in two courses provided in the original construction. Primary highways will have 0.3 ft in two courses for roadway widths of 40 and 44 ft, and 0.2 ft in one course for roadways 34 ft wide. Secondary roads will be given 0.2 ft of bituminous surfacing of either plantmix or roadmix, depending on the traffic count. All bituminous pavements are constructed full width from shoulder to shoulder.

Base courses immediately below the bituminous surfacing will consist of 0.4 ft of ¾-in. maximum base in two courses for the wider roads, and 0.3 ft in a single course for the 24- and 28-ft roadbeds. Other subbase and base courses needed will be governed by the total thickness to be provided and whenever possible will be of pit run material but having characteristics very similar to the high-type base material for plastic index and sand equivalent. An endeavor is made to always provide a free-draining material in the base courses, including any select materials used. The base courses are also constructed full width. If the select material or base possesses any plasticity, R-value determinations are made to determine the suitability for use. The adoption of the sand equivalent test in the Idaho specifications in 1956 has aided materially in providing apparently free-draining material.

An example of the soil evaluation presently furnished the Design Engineer is shown in Figure 8. This evaluation sheet includes the group index number as a part of the soil classification by noting it in parentheses. The empirical soil number is reported, as well as R value and expansion pressure. A total thickness of pavement structure is recommended for the traffic index shown at the head of the sheet and is generally based on R value or expansion pressure. However, if the soil number gives a greater total thickness for an area subject to frost damage, the greater thickness is used. The design engineers and district materials engineers are getting together and establishing the thickness to be provided for frontage roads, ramps to interchanges, and separation structures. This information is included in the project soil report, which covers the soil survey, soil profile, sub-subgrade treatments, and total flexible pavement design thickness for the project.

EVALUATION OF DESIGN PROCEDURES

Idaho has been using R value as one basis of design since 1950 and has been using the Hveem stabilometer for determining the stability of bituminous mixtures since 1938. Without the kneading compactor, however, the results obtained in stability tests and R-value tests in all probability are misleading. The combination of the compactor and the stabilometer makes a most useful and rapid instrument for testing all material used in the construction of a pavement structure from the subgrade up to and including the bituminous surfacing. Much greater confidence is now placed in the R-value method of design rather than on the empirical soil number. The resistance value test is considered no more subject to experimental error than CBR. It is sufficiently rapid to permit at least four values to be determined for each soil sample, permitting the selection of an R value from a curve rather than from a single determination. It is felt that this test and the method of application will do much toward reducing the number of failures of subgrade, base, and surfacing on Idaho roads.

An attempt is now being made to measure the permeability of granular soils when testing for expansion pressure. The rate of flow through the sample is a variable and by means of measuring time and drop in the water level within the mold, a value for permeability can be computed. This value may not be as accurate as desired, but does appear to indicate the permeability within a reasonable range. It has been found that some A-3 soils have very low permeability, whereas permeabilities in apparently dense-graded material have been very rapid. Not many data are available as yet, but this test is being conducted on any soil considered possibly permeable, as well as on all select borrow and base materials tested for R value.

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   (c) Test Method T-8-54, "Standard Method of Testing Soil for Resistance Value."
   (d) Test Method T-9-54, "Standard Method of Testing Bituminous Mixtures for Relative Stability."
   (e) Test Method T-15-54, "Standard Method of Test for Degradation of Aggregates."

(f) Test Method T-26-56, "Standard Method of Evaluating Subgrade Soils for Total Flexible Pavement Thickness."

5. Hveem, F. N., "Ideas and Current Problems in Pavement Design." Presented at Seminar on Asphalt Paving Technology, Univ. of California, Berkeley (July 1957).

HRB:OR—210

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