HIGHWAY RESEARCH BOARD

Bulletin 136

Flexible Pavement Design In Four States

National Academy of Sciences—National Research Council

publication 427

HIGHWAY RESEARCH BOARD

Officers and Members of the Executive Committee
1956

2101 Constitution Avenue — Washington 25, D. C.

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Bulletin 136

Flexible Pavement Design In Four States

PRESENTED AT THE
Thirty-Fifth Annual Meeting
January 17–20, 1956

1956
Washington, D. C.

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Experience with Flexible Pavements in Maryland

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This report is concerned principally with the historical development of the design and construction of flexible pavements in Maryland and with the structural adequacy of pavements built according to the provisions of the method of design currently in use.

This method utilizes CBR values of the subgrade soil as a basis of determining the thickness of subbase material necessary as an interfacial layer on the subgrade beneath either of two standard sections. One of the standard sections is for traffic of 2,000 vehicles or more per day and consists of 10 inches of macadam base and 3½ inches of asphaltic concrete; the other is for traffic less than 2,000 vehicles per day and consists of 8 inches of macadam base and 2 inches of asphaltic concrete.

The service behavior of the majority of pavements built in the state during the past several years using the method has been satisfactory.

DESIGN

THE design method followed in connection with flexible pavements in Maryland is not unique, but rather has been developed by evolution and investigation of current research. It makes use of the CBR test as a means of determining the overall thickness of pavement.

In common with many states, Maryland built many miles of flexible pavement during the early days of its State Roads Commission. Due to the availability of suitable aggregate over a large portion of the state, these early pavements were constructed of 5 inches of waterbound macadam, highly crowned. This of course was a section based on judgment, and one can hardly say that any rational design entered into the picture. They were good roads for their day, however, and pictures of early macadam road building in Maryland have found their way into highway texts and Public Roads murals.

With the increase in the intensity of traffic, and weight of motor trucks, it became apparent that a heavier section, and one whose surface was more resistant to abrasion and ravelling was needed. Again based on judgment, a section of 8 inches total thickness came into use. This consisted of the same 5-inch layer of waterbound macadam, with a 3-inch layer of bituminous penetration macadam placed on top. The surface of the penetration macadam was double sealed. This section came to be regarded as the standard for flexible-type construction, and was used until the late 1930's. During this period Maryland, along with many other states, began making detailed soil investigations and reports for each project. This work was under the direction of the Materials Division and is referred to in more detail later in this paper. After the advent of the detailed soil investigations, the practice of placing selected granular subbase material beneath the standard section, where conditions indicated it necessary, was adopted.

The World War II years, closely following the period previously mentioned, brought practically all highway construction to a halt, particularly after the military access road program was completed. However, research in flexible pavement design was greatly accelerated during this period.

In common, with the engineers of most highway departments, Maryland used this slack period to look into its practices in many fields, and the field of flexible pavement design and construction was realized to be one needing close attention. The work of the Corps of Engineers, the Bureau of Public Roads, the Civil Aeronautics Administration, the Bureau of Yards and Docks, and others, was carefully considered. It seemed reasonable to begin the basic study by an evaluation of some of the same fundamentals which most other states have considered. These fundamental elements are load, area of contact, distribution, and bearing capacity of the subgrade. Review of the development literature in this field led us to consider the following assumptions: (1) elliptical contact areas of single and dual tires; (2) a 1:1 distribution of the load to the subgrade material; (3) an equivalent uniform pressure equal to half the maximum bearing capacity

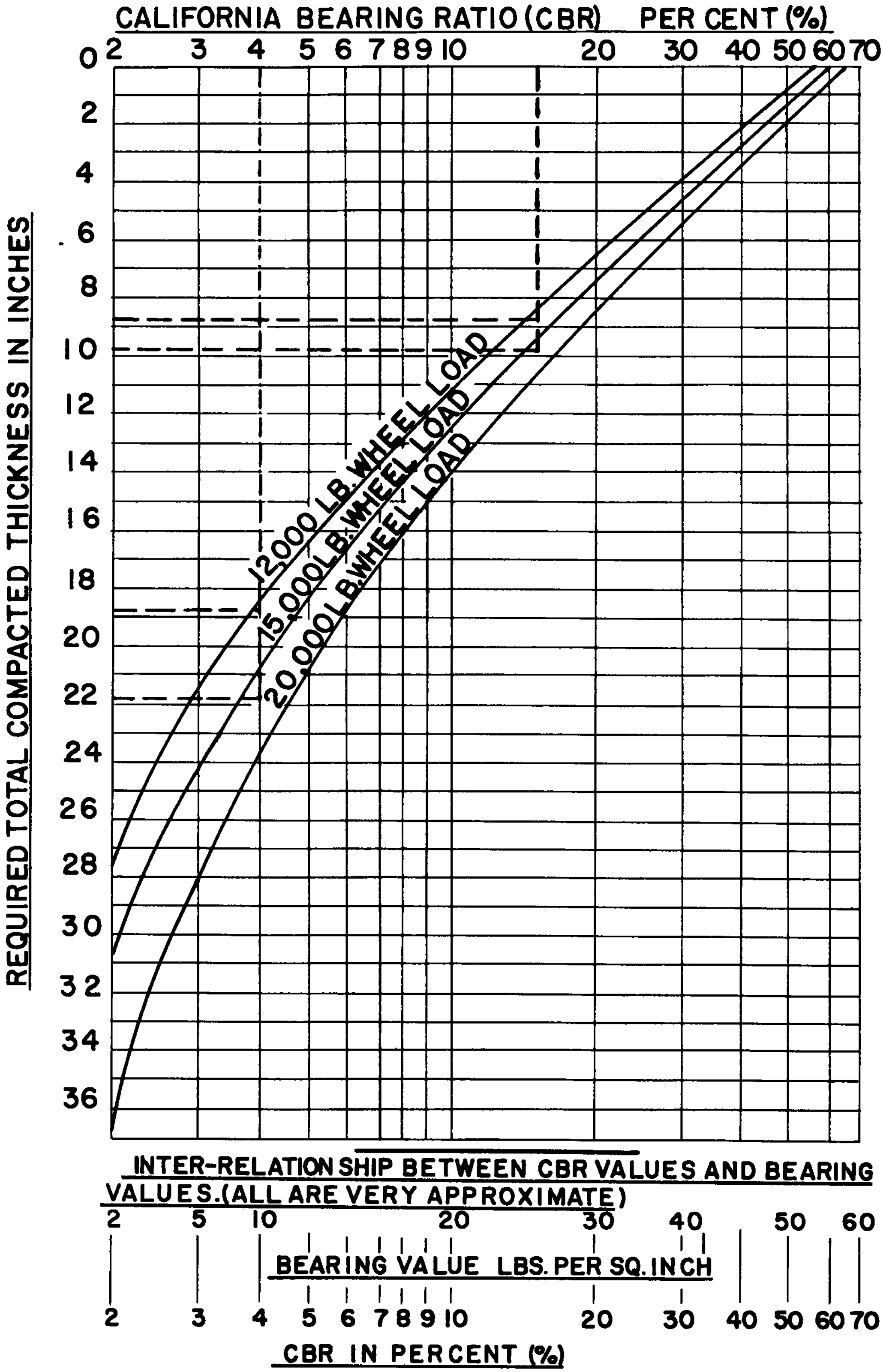


Figure 1. Flexible pavement - design chart.

TABLE 1

DEFLECTION DATA SUMMARY

22,400-Pound Rear Axle Load (Deflection Values in .001 Inches)

Route Number	Time of test 1955	Wheel Path[a]	Deflection: No. Tests	Deflection: Mean	Deflection: Standard Deviation[b]	Remarks
US 40 W	Spring	OWP	269	32	15	
		IWP	276	22	10	
		BWP	545	27	-	
	Fall	OWP	276	25	-	
		IWP	278	18	-	
		BWP	554	21	-	
US 240	Spring	OWP	148	44	18	
		IWP	148	37	15	
		BWP	296	40	-	
	Fall	OWP	159	32	-	
		IWP	158	28	-	
		BWP	317	30	-	
US 40 E	Spring	OWP	138	70	28	Stage construction
		IWP	138	67	24	Surface treatment only.
		BWP	276	69	-	
	Fall	OWP	138	26	-	AC surface course placed in
		IWP	136	25	-	late summer and fall of 1955.
		BWP	274	25	-	

Note: Deflection values are for outer lane only.

[a] OWP - Outer wheel path.
IWP - Inner wheel path.
BWP - Both wheel paths combined.

[b] 68 percent of test values fall within ± one standard deviation of the mean.

of the subgrade; and (4) the maximum bearing capacities of various subgrade materials.

It was definitely realized that heavy axle load was going to be one of the major points of consideration in Maryland. About 15 years ago legislation was enacted in Maryland which allowed the gross weight of a single axle to go up to 22,400 pounds. Subsequent legislation has provided another class of carriers to be included in the category of "Dump Service Registration." This legislation provides for a gross weight of not more than 40,000 pounds for two-axle vehicles, nor more than 65,000 pounds for three- or more axle vehicles. These registrations are issued to dump trucks hauling loose material in bulk. Vehicles so registered may be operated within a radius of not more than 40 miles of the point of pick up. Considering the many quarries and sources of bank run gravel, etc., within the state, and the allowable 40-mile radius, it can be seen that a very considerable portion of the state highway system can be subjected to the loads from this class of registration. According to the Traffic Division of the Commission the rear axles of many of these dump trucks carry a load between 28,000 and 31,000 pounds. In addition to the consideration of axle loads, Maryland's geographical position places it along the heavy trucking routes of the Eastern Seaboard region and the high volume of truck traffic must necessarily be a prime consideration in the selection of the flexible

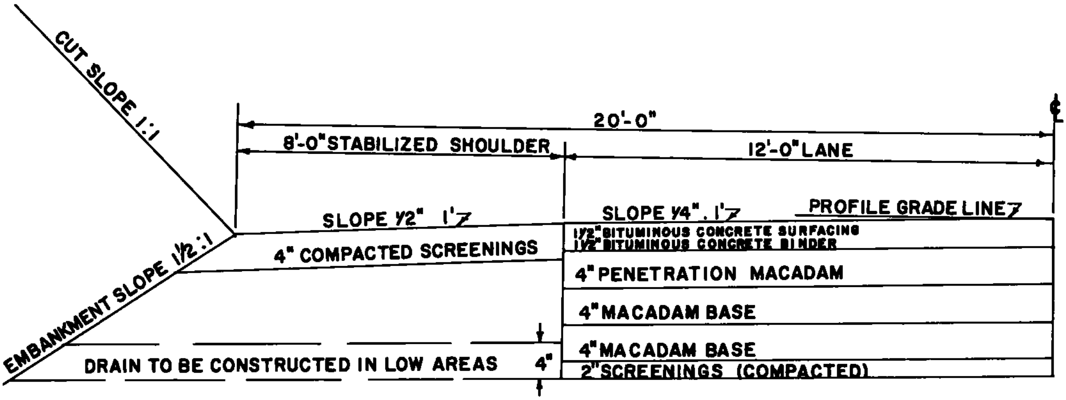


Figure 2. Partial typical section of improvement. US 40, west of Frederick toward Hagerstown.

section for any route. It has been noted that some research studies have been made concerning the number of load repetitions required to produce failure in flexible pavements of various depths. However, no rational basis has been found for relating these two factors. Consequently, Maryland's considerations along this line are still pretty much a matter of practical judgment.

The effect of impact loads on flexible pavement is also a subject which commands no uniform thought. While it has been observed that maximum pavement deflections usually occur at low speed, it nevertheless seems reasonable to include at least a small allowance for impact effect. The Maryland Roads Commission selected a value of 10 percent of the static load. This factor applied to the axle loads previously noted gives wheel loads of 12,300 pounds for ordinary carriers and 16,500 pounds for the dump truck classification, considering 30,000 pounds as an average rear axle load in this category.

Although frost penetration varies appreciably throughout the state, severe winters can cause a penetration of such depth that it must be considered a significant factor in the design of pavements. Frost conditions in the central and western regions are sometimes quite severe. An average annual penetration of 24 inches occurs in the western portion of the state, with a maximum of 36 inches occurring in this region during severe winters. A report published some years ago, moreover, indicates that the remaining portions of Maryland may be subject to maximum depths of frost penetration varying between 18 and 35 inches.

The development of pavement thickness formulas based on the first three of the assumptions mentioned earlier have been published previously. The formulas used were presented in a paper entitled "The Problem of Flexible Pavement Design," by A. T. Goldbeck, published in the Crushed Stone Journal, June, 1948.

A formula for the thickness of flexible structure necessary to support a load on dual

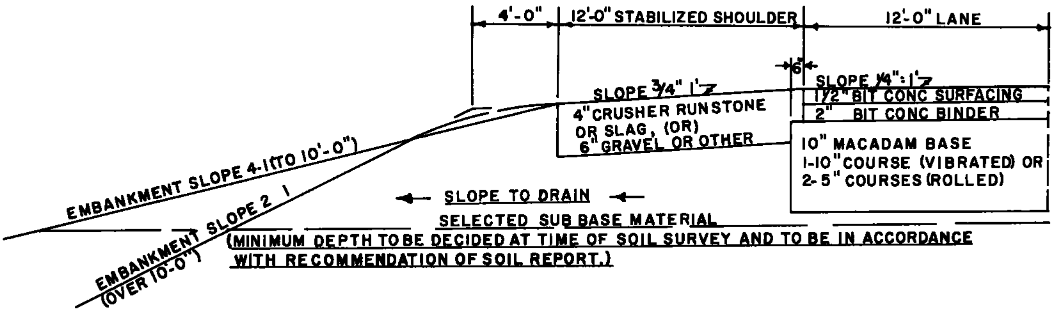


Figure 3. Partial typical section of improvement. Standard macadam section for traffic counts of 2,000 or more vehicles per day.

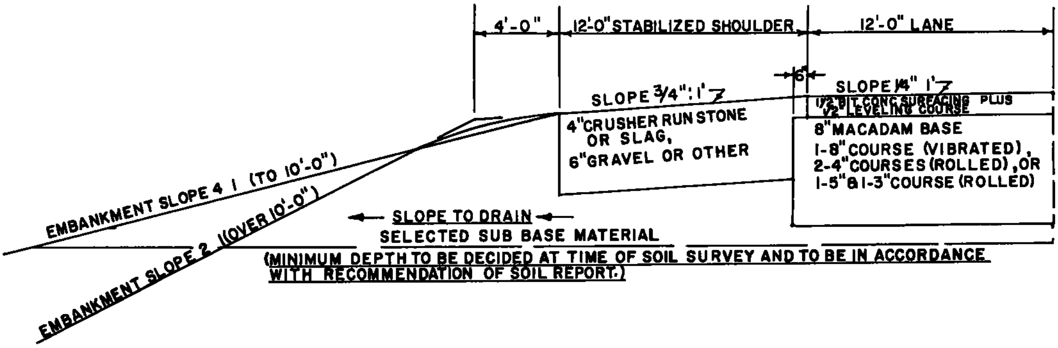


Figure 4. Partial typical section of improvement. Standard macadam section for traffic counts up to 2,000 vehicles per day.

tires is given in this article as follows:

$$T = \sqrt{\left(\frac{B}{2\pi}\right)^2 + C} - \frac{B}{2\pi}$$

Where $B = 2S + \pi (L_1 + L_2)$

$$C = \frac{Pk}{M\pi} - \frac{2SL_1}{\pi} - L_1 L_2$$

where A = area of equivalent uniform subgrade pressure
P = wheel load
U = equivalent uniform pressure over area A
M = maximum subgrade pressure and also the bearing value of the subgrade
S = center to center spacing of dual tires
L_1 = half major axis of tire contact area
L_2 = half minor axis of tire contact area
$k = \frac{M}{U}$ assumed to be = 2

or assuming $k = 2$, and $L_1 = 2L_2$

$$B = 2S + 3\pi L_2$$

$$C = \frac{2P}{M\pi} - \frac{4SL_2}{\pi} - 2L_2^2$$

Many subgrade bearing values were thus investigated, but for purposes of illustration the example here is limited to one very low value, namely 10 psi. Using this value for the 12,300-pound wheel load, an overall structure thickness of about 19 inches is indicated. If the bearing value of the subbase is assumed as 25 psi., the combined thickness of the base and surfacing layers would be indicated as about 10 inches. Considering the heavier wheel load of 16,500 pounds and using appropriately larger tires, for the same bearing values previously mentioned, an overall thickness of about 22 inches is indicated, and a combined base and surfacing course thickness of about the same, or 10 inches.

After a thorough study of the problem, the conclusion was reached that insofar as total thicknesses were concerned, the CBR curves as originally developed from the work done in California would suit the present purpose admirably. (See Figure 1) We did not feel satisfied, however, with the depth of macadam base and asphaltic concrete surface indicated by these curves for the very heavy loads to which our highway system is subjected. Naturally we had a reason for this concern. Our experience over a long period indicated that the former generally used section of 5-inch waterbound and 3-inch penetration macadam had not been entirely satisfactory under heavy traffic, even though, in many cases, the subgrade conditions were good. Practically all of our heavily used macadam roads had required rehabilitation through the years, and many of them cored

Figure 5.

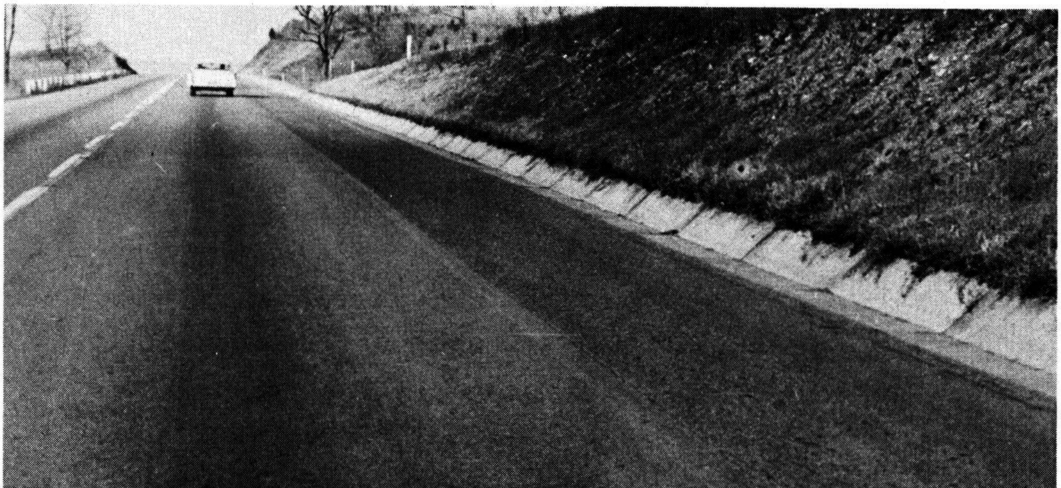


Figure 6.

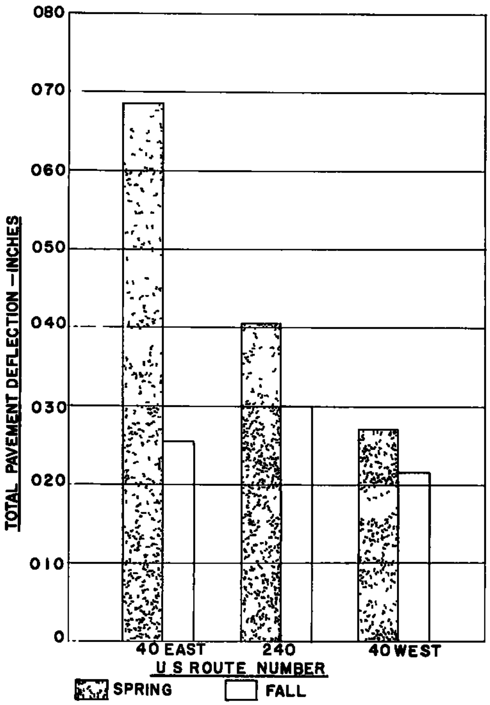


Figure 7. Comparison of mean deflections observed on the various pavements in the spring with those of the fall. (1955)

for investigation have shown combined thicknesses of original macadam, and rehabilitation courses of 12 inches and more.

The first high type flexible pavement project scheduled in the post war period was along US 40 west of Frederick, Maryland in the western portion of the state. This is a route subjected to heavy trucking, the possibility of the operation of dump truck vehicles is present, and frost conditions can be quite severe. After a thorough consideration of the detailed investigation noted earlier, a very substantial section was selected for the pavement on this route. The load bearing components as shown in Figure 2 were as follows: two 4 inch layers of waterbound macadam, one 4 inch layer of penetration macadam and 3 inches of asphaltic concrete, placed in two courses. The two courses of asphaltic concrete consisted of 1½ inch binder and 1½ inch surfacing. The pavement along this section of US 40 has been very closely observed during the 10 years which it has been in service. Later in this paper we will comment in detail concerning the performance and the maintenance costs of this pavement. In general its performance has been so good that we considered it reasonable to use this design as a guide for later flexible pavements.

Several years after the completion of the US 40 project noted above, the state embarked on its first post war accelerated highway building program. At present it is engaged in an even larger construction and reconstruction program which covers a 12 year period that began January 1, 1954. The conclusion was reached that it would be advantageous to develop standard sections for flexible pavements, for cases where our economic studies indicated that this type should be built. We knew, however, it would be necessary in many cases to supplement such standard sections with selected subbase material, the thickness of which would be determined by our materials division from their studies of the prevailing soils. The use of a standard section supplemented by varying thicknesses of subbase has been described in one of the Highway Research Board publications dealing with flexible pavements as an inverted method of design. All of the data gathered during the slack period of World War II, and our experience with the US 40 pavement was carefully considered.

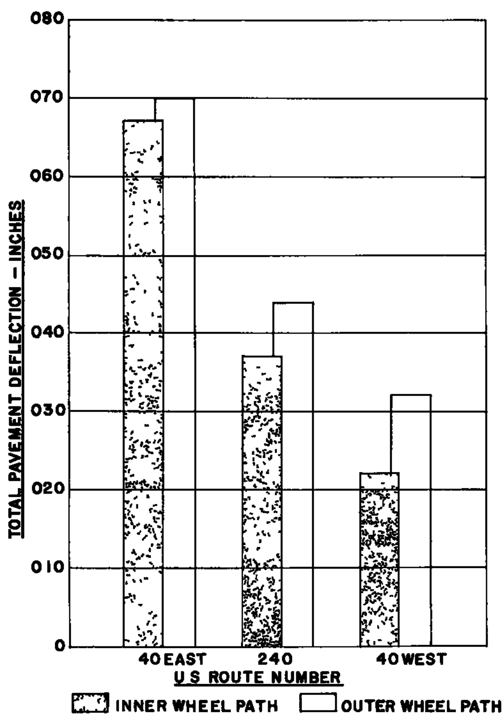


Figure 8. Mean deflections observed on various pavements in the spring of 1955.

After additional study and evaluation participated in by engineers of our districts, and the construction, maintenance, materials, and design divisions, we selected the following two standard sections:

(a) For highways carrying 2,000 or more vehicles per day, a 10 inch macadam base course plus 3½ inches of asphaltic concrete surfacing placed in two courses, a 2 inch binder and a 1½ inch surfacing course. (See Figure 3)

(b) For highways carrying less than 2,000 vehicles per day, an 8 inch macadam base course, plus 2 inches of asphaltic concrete placed in two courses, a leveling course of an average thickness of ½ inch and a 1½ inch surfacing course. (See Figure 4)

It is pertinent to elaborate a bit on the detailed soil survey and analysis which has been mentioned previously. Borings are made at an average spacing of 300 feet center to center on the right, left and/or centerline of the project. In cut sections they are carried to a minimum depth of 3 feet below subgrade. In all areas they are made to a minimum depth of 3 feet below the original ground surface. In fills at least one boring is carried to a depth below the original surface equal to the height of fill. Borings closer than 300 feet center to center are made if nonuniform conditions prevail. Boring equipment includes hand augers, gasoline powered augers, and jeep mounted drills. Additional field data pertaining to rock conditions, water conditions, and swamp comditions are recorded. Complete gradations by sieve and hydrometer methods are determined in the laboratory. The following tests are also run as a routine procedure: field moisture equivalent, liquid limit, plastic limit, shrinkage limit, and Proctor compaction. Californing Bearing Ratio Tests are conducted on selected samples on all representative soil types encountered on the project. The report from the materials division to the design division outlines in detail the results of the soil survey and analysis and recommends the thickness of subbase in conformity with the CBR values of the subgrade soils encountered.

The present design procedure has now been followed for approximately the last six years. While this period is relatively short for a long range evaluation, we feel that we have been obtaining very good results. In the section of this paper dealing with the performance of flexible pavements, we will discuss this point in more detail. Also in the section dealing with construction, we will comment on the possibility of introducing additional types of construction for flexible bases.

CONSTRUCTION

We do not intend, nor consider it pertinent, to give any long and detailed descriptions of complete sequences of flexible pavement construction. We will limit ourselves to a few comments on macadam construction.

Considered from the historical standpoint, we have already mentioned that Maryland built a considerable mileage of macadam roads in the early years of this century, and that for their day they were excellent, enduring the rigorous weather conditions of all portions of the state. Our advisory engineer, who experience spans the period 1910 to the present, has shed some interesting light on this early period. Crowns were high, the pavement slope being ½ inch per foot for grades up to about 5 percent, and ¾ inch per foot for grades over 5 percent. Overall roadbed widths were 24 feet, the travelled way being usually 12 feet to 14 feet, except in the Baltimore or Washington area, where 16 feet was used. The rolling, choking, and watering operations were very similar to these same operations, if resorted to today. Of course, in those days, much use was made of hand labor, which cannot be afforded to the same extent today. It is interesting to note that a road finished after November 1 was not accepted from the contractor until the following construction season; also that a base was well compacted and acceptable if it gave satisfactory metallic "ring" when the Chief Engineer drove over it in a horse drawn, metal tired conveyance; and any "mud intrusion" which showed up through the courses was required to be removed by the contractor and replaced with new base construction.

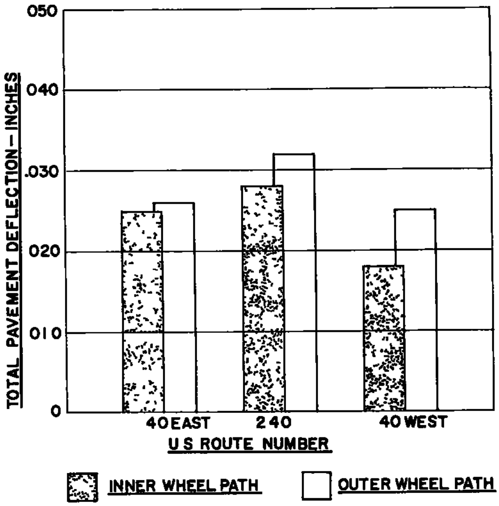


Figure 9. Mean deflections observed on the various pavements in the fall of 1955.

Today, pavements of the flexible type are built when indicated to be justified by detailed economic analyses, including first cost, salvage value, maintenance, interest, etc. Traffic forecast for a period twenty years hence, gives a determination as to which of the two standard sections should be used. The detailed soil borings, laboratory analysis, and report and recommendations from the Materials Division determine the need for selected subbase material, and the depth to which it should be placed. The preliminary field investigation is made as soon as the preliminary grade line has been set. This field investigation and the office conference which follows, is an important phase of the preconstruction period. Representatives of the Design and Materials Divisions participate. Preliminary location of subdrainage, grade line with respect to water table.and other pertinent field conditions, optimum locations for surface drainage structures, etc., are set at this phase of development.

Considerable use is made of a 2 inch compacted layer of stone screenings between the subgrade or subbase and the macadam base course. This is the usual insulation layer, and is always used where the subbase material consists of bank-run gravel. However, where a crusher run stone or crusher run slag type of subbase is used, the stone screenings course is not considered to be necessary.

The use of subgrade drains (so called shoulder drains) is most important to this type of construction. The state installs them at 100 foot intervals, except in sumps where a total of 10 or 12 are placed 25 feet apart.

The traditional construction methods of loose spreading, rolling, choking, and watering still may be followed, according to our latest specifications. A maximum compacted thickness of 5 inches may be placed by this method. Recently we have allowed the use of vibratory compactors in the construction of macadam bases. A minimum of 5 inches compacted thickness, up to a maximum of 10 inches compacted thickness may be placed in one course by this method.

After the compaction and dry choking by either the rolling or vibratory method is accomplished, watering and additional applications of screenings follow until a well finished surface is produced. This is evident by the absence of voids, and the absence of an excess of loose screenings in any spots. Deviations exceeding ½ inch from the true transverse template must be corrected. Likewise, longitudinal deviations greater than ½ inch in 10 feet must be corrected.

Regardless of what type of compaction is used, we are convinced that a construction crew and inspectors with a real interest in the finished product, are absolutely necessary to produce top quality work. Also, hand forking, picking, and casting are just as essential to obtain the best results today, as was the case in the early days of road construction.

Although this type of flexible pavement construction has been standard for about 5 years or so, we do not consider that its continued use is mandatory, and that we cannot change some of its components if conditions warrant. For instance, some of our flexible pavements have been built using stage construction methods. The macadam base of a project recently built in this manner was surface treated and a great deal of damage resulted from the penetration of water through this temporary surface into the underlying base. We are now considering the use of an alternate type of section in the event a project is to be built by stage construction methods. The upper layer will consist of 3 inches of bituminous penetration macadam with a double seal treatment as the temporary wearing surface.

We are also much interested in the plant-mixed, dense-graded aggregate base which several of our neighboring states have used to such good advantage. Our latest specifications include this type of base construction. Briefly, it consists of coarse aggregate, fine aggregate, calcium chloride and water, plant-mixed, mechanically spread, and compacted with rubber tire rollers. Although we have not built any roads of this type as yet, we plan to let a few pilot jobs in the near future. If the prices seem to be favorable and the product satisfactory, we will most likely use more and more of it as time goes on. It is quite possible that it will supplant our present standard sections, at least in certain areas of the state, where material supply conditions are favorable.

PERFORMANCE

In evaluating the performance of various flexible pavements to determine if our standard sections are reasonable, we have rather carefully considered many roads built throughout the state since the early 30's. Although it is only since the World War II period that flexible pavements in Maryland have been designed for heavy duty service, we felt that it would be helpful and necessary to observe many examples of our earlier construction. We have obtained very good service from the older roads, but in almost every case maintenance operations have been necessary which resulted in a significant increase in their total thickness. This in effect is the equivalent of stage construction, a process to which, as we all know, the flexible type of construction is particularly well adapted. Generally speaking the 5 inch macadam base and 3 inch penetration macadam surfacing did not prove to be a sufficiently thick section to withstand heavy trucking service, even though built on good subgrades. We have in mind one example of a major route over mountainous terrain in the western portion of the state. Although the total volume of traffic is not unusually high, the percentage of trucks is a good bit above the average found in most parts of the state. Rutting of the pavement has been pronounced, and before long it will be in need of resurfacing. We have found that on relatively heavily travelled roads of the above type, rehabilitation was necessary after a period of service of perhaps six years. We have cored a number of highways of this type throughout the state. Some of them were built in the 30's and some in an even earlier period. In many cases we have found 12 inches of substantial road metal, consisting of various combinations of waterbound macadam, penetration macadam, cold mixes, and hot mixes. Our evaluation of the performance of these roads has been limited to the history of their behavior and the study of cores taken from them.

As noted earlier, our experience and observation of the behavior of older pavements let us to select the rather substantial section for US 40 west of Frederick. The behavior of this road has been observed diligently, as we considered it a pilot job which might lead us into some standardized sections. It has now been in service for close to 10 years and its performance has been excellent. There are only two localized areas where any roughness has developed and this is quite minor. Like many states, we did not always break our maintenance costs down into the separate items of surfacing, shoulders, ditches, etc. However, since June, 1949 we have kept itemized records of maintenance costs. The records show that, for this Route 40, about 16.5 miles long, the average annual surface maintenance cost per mile for the four-year period from June 30, 1949 to June 30, 1953 was $58.00. This amounts to about 7/10 of a cent per square yard per annum. The selection of the standard sections which have been described earlier in this paper was based, in part, on these observations. (See Figures 5 and 6)

Within the past year we were fortunate in being able to arrange, in cooperation with the Bureau of Public Roads, for the conduct of two series of deflection tests on several of our modern designed flexible pavements. The first series of tests was made early in the spring and the second in November.

Three pavements were selected for the tests in the vicinity of Frederick, Maryland: the ten year old project to the west on US 40; a 16 mile section of the New Washington National Pike pavement, US 240, to the south toward Washington; and a portion of the new pavement on the Baltimore National Pike extending eastward toward Baltimore, 12 miles in length. The total traffic on US 40, both east and west of Frederick, totals about 6,000 to 8,500 vehicles per day. That along US 240 south of Frederick totals some 5,000 vehicles per day, but will most likely increase greatly as the route is completed to the Washington, D. C. area. Commercial vehicles account for about 20 to 30 percent of the total traffic.

In the tests the deflection of the pavement was measured at selected points under an 11, 200 pound moving wheel load with the Benkelman beam device. (1) This device records the vertical movement of the pavement surface midway between the dual tires of a loaded truck wheel as the load approaches and leaves a given point. Two of the beam devices were used simultaneously to measure the deflection under both sets of rear wheels, with the center of the outer dual wheel positioned approximately 18 inches from the pavement edge. About 20 locations were tested per mile.

The initial series of tests was made at a time when the condition of the subgrade was considered to be adverse, and the second series at a time when the condition was considered to be more favorable. The results of the tests are presented in Table 1 and are shown graphically in Figures 7, 8, and 9. They may be summarized as follows:

1. On a basis of all the tests, both wheel paths combined, the deflection of the pavement in the spring period was 26, 35, and 169 percent greater than in the fall for the US 40 West, US 240 and US 40 East projects respectively. (See Figure 7) The values of 26 percent and 35 percent represent what we believe to be a more or less typical decrease in the indicated ability of a pavement of this type to support load in the spring as compared to the fall. (2) It should be noted here that the US 40 East project was built by stage construction methods. When the spring deflection measurements were made, only a surface treatment had been placed on the macadam base. This, no doubt, accounts for the high deflections found at this time. Prior to making the fall tests, the final asphaltic concrete surfacing had been placed, and the deflection values were considerably less.

2. For the tests made in the spring period, the average deflection in the outer wheel path was about 4 percent greater than in the inner wheel path for the US 40 East pavement, 19 percent for the US 240 pavement and 45 percent for the US 40 West pavement. (See Figure 8) Comparable values for the series of tests performed in the fall period are 4 percent, 14 percent and 39 percent respectively. (See Figure 9)

3. As shown in Figure 9, little difference in the mean deflection of the three pavements was found in the outer wheel path for the fall period of testing. In the inner wheel path there was not much difference in these values for the US 40 East and US 240 pavements; that in the inner wheel path of the US 40 West pavement was, however, considerably less, for which we have no explanation.

4. That considerable variability exists in the indicated load supporting capacity of the three pavements is shown by results of a statistical analysis of the spring deflection data. (See Table 1) For example, while the mean deflection of the US 40 West pavement in the outer wheel path was 0. 032 inch, 68 percent of the total of 269 measurements ranged from 0. 017 to 0. 047 inch. These values for the US 240 pavement are 0. 044 inch, and 0. 026 to 0. 062 inch; for the US 40 East pavement they are 0. 070 inch, and 0. 042 to 0. 098 inch.

The WASHO Road Test report contains an analysis of deflections correlated with satisfactory and unsatisfactory pavement performance. The conclusion of this analysis is that 0. 045 inch deflection is satisfactory for warm weather periods, and 0. 030 inch is satisfactory for cold weather. The report states that these values do not necessarily apply to other pavements, or to pavements of greater age.

The mean deflection on one of the new roads, US 240, agrees quite well with these values, while the mean value for the older project, US 40 west of Frederick, is considerably smaller.

We believe that the results of the deflection tests indicate in a general way that pavements being built using our selected design method will perform satisfactorily. However, the variability in indicated load supporting capacity as shown by statistical analysis of the deflection data is such that we do not feel that any reduction in the pavement structure would be justified. Apparently some factor of safety is necessary to ensure satisfactory performance of such flexible pavements as we have built, or are building in Maryland.

It is anticipated that we will continue our studies of the performance of pavements in service in Maryland, and that, as more data are accumulated, we will be in a better position to decide on the merits of the approach to this problem.

ACKNOWLEDGEMENT

The assistance of Stuart Williams, Highway Physical Research Engineer, Bureau of Public Roads, in the conduct of the deflection tests, and the analysis of the data is gratefully appreciated.

References

1. The WASHO Road Test, Highway Research Board Special Reports 18 and 22.
2. Public Report of the Highway Research Board Committee on "Load Carrying Capacity of Roads as Affected by Frost Action."

Colorado's Flexible Pavement Design Method

R. E. LIVINGSTON, Planning and Research Engineer
Colorado Department of Highways

●THE opportunity to report to the Highway Research Board on the evolution of our design method is one that is appreciated by the engineering staff of the Colorado Department of Highways. Actually, the method was originally reported to the 1947 Annual Meeting of the Highway Research Board and was subsequently published in the Proceedings of that year. Any who are interested in a detail review of the method under discussion have access to that publication.

Briefly, to lead-in to the following discussion, the Colorado design method evaluates (a) the capabilities of the normal basement soils, to sustain loads, when they are in different degrees of saturation; (b) the anticipated traffic volumes for a period 20 years hence; and (c) the damage to the pavement structure that is probable from the frost potential of the soils over which the pavement is to be placed. With these fundamentals determined on an empirical evaluation, a thickness is determined from a series of five curves. The curves used are shown in Figure 1. Actually, Curve A relates to very light volumes of traffic combined with low moisture and frost potential and Curve E is at the extreme of the heavy volumes combined with high moisture and frost potential.

The only difference between the present design chart and the original is in the elimination of a group index value which was shown on the bottom of the chart. At the time of the original preparation, we attempted to correlate Group Index and California Bearing Ratio. This proved to be groundless as a generalization, hence the elimination of the Group Index value from the chart.

In addition to the elimination of the group index value from the design chart, we have made another major change for the soils of the A-1 and A-3 classifications. Their evaluation, which was originally obtained by CBR method, is now determined by the stabilometer equipment developed by Hveem. This change was made because we feel that more consistent values are developed for granular materials by the stabilometer than by a direct sheer test.

Another fundamental change is also tied to the use of stabilometer values. After the total thickness of the pavement structure has been determined from the design chart (Figure 1), it is assumed to be the "gravel equivalent" of the California design chart shown as Figure 2. This "gravel equivalent" is then used in combination with the California design chart to develop the balance of the pavement structure. As a side note, it should be explained that our acceptance of the Hveem stabilometer for evaluating granular materials was considered for a long time before adoption. In prior years, we had assumed that granular materials of the same sieve analysis and having similar Atterberg limits were actually equal materials and could be used from a design standpoint as having identical characteristics. Field performance did not substantiate this assumption. Literally, the assumption would actually mean that crushed materials and rounded water-washed materials would have the same amount of stability. In addition, it presumed that the minus 200-mesh material was always identical and that in combination with the granular materials it would produce identical surfacing materials. Actually, we have long known that the minus 200-mesh material can vary widely in its capabilities for altering the performance of soil-aggregate mixtures. This is easily explained by the fact that the minus 200-mesh material can be anything from a rock flour or a lime stone dust to a highly expansive clay. The actual potential that these widely separated materials have to alter the performance characteristic of a mixture, of which they are a part, is widely recognized.

Now to discuss the merits of the design method on the basis of our experience. Simply stated, it is so far superior to what we were doing previous to 1947 that all of the engineering personnel of the department have accepted it without question and only disagree on some of the minor details regarding application. Each of us feels that a method which gives a definite answer to the pavement thicknesses to be used for varying conditions is a necessity. One which provides uniformity of application and ease of handling in the field is highly desirable. We don't believe that anyone in the department would

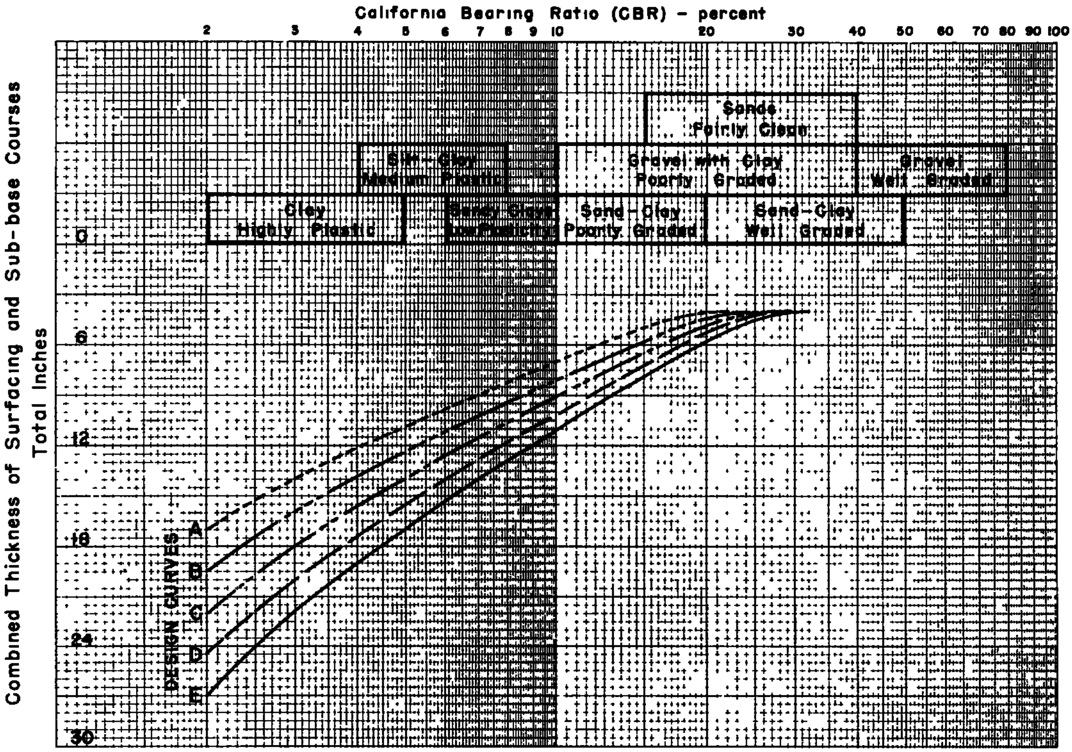


Figure 1. Design chart for thickness of surfacing and subbase courses.

go back to the method where variations of 10 to 12 inches in thickness for identical operating conditions would be possible. On the other hand, we do not want to leave the impression that every job planned under this design method has been a masterful success and that we have had no failures.

To have a look at the procedural angle, we will go to a description of the method of carrying out a typical project. At the time the ground survey, giving center line profiles and topography, has been plotted, a set of the preliminary plans are handed to the central laboratory as an automatic order to proceed to obtain the soil and material surveys. If heavy drilling equipment is necessary, the central laboratory is the only one with equipment to undertake the soil survey and they then proceed to schedule this work. The results of the work are submitted to the field district, in which the project is located, and to the Surveys and Plans Engineer who is responsible for design. If the profile is uniform and no deep holes are required, the soil survey information is often acquired by the field district. As a minimum, auger holes are driven at the beginning and ends of cuts and in the center of the mass. In uniform profiles, the soil samples are taken at intervals not to exceed 1,000 feet. Intermediate samples are lifted at anytime there appears to be a soil change. The depth of the borings are not less than 3 feet below the profile grade. This minimum has been established to insure that the soil samples extend far enough into the basement soil to provide necessary data for design.

Using the soils information from the central laboratory, which includes frost potential established from the soil samples; traffic data supplied by the Planning and Research Division, from their traffic surveys; and, frost penetrations from field surveys, an evaluation is made which automatically selects a design curve or curves for the project. The use of multiple curves can and does occur because of the difference in potential for soaking of the pavement structure, including the basement soil. The potential for soaking varies for the environment changes inherent on any job. As an example, through cuts on flat grades are always subject, in snow country, to continuous infiltration of surface moisture. High fills on steep grades, which drain very rapidly, provide very little opportunity for surface moisture infiltration. Obviously, the potential of the two conditions for soaking is essentially different. The design curves that would be used under the two conditions would thus be different.

Colorado predesignates pits from which the contractor is expected to obtain materials for use in construction of the pavement structure. The granular materials are selected from designated pits on the basis of Hveem criteria for subbase, base course and wearing course materials.

During construction, the basement soils are moved into a position called for in the design, and, if there is any apparent variation from the design presumptions, sand equivalent tests are run to find out if a change in thickness is desirable. Some correlation CBR's are run on the constructed foundations to assure that the design presumptions are being carried out. Such correlation data is obtained only when the district personnel feel that it is required, based on the other physical tests.

With 8 years of experience, certain performance data have become available and a very brief discussion of it will be attempted at this point. Our method of rating performance is related to our annual inventory of road conditions, published as a Rural Sufficiency Rating Study. Special evaluations are made in 22 different test areas where physical data on the basement soils and on construction materials are available. The sufficienty rating, used in combination with this information, is a fairly effective tool

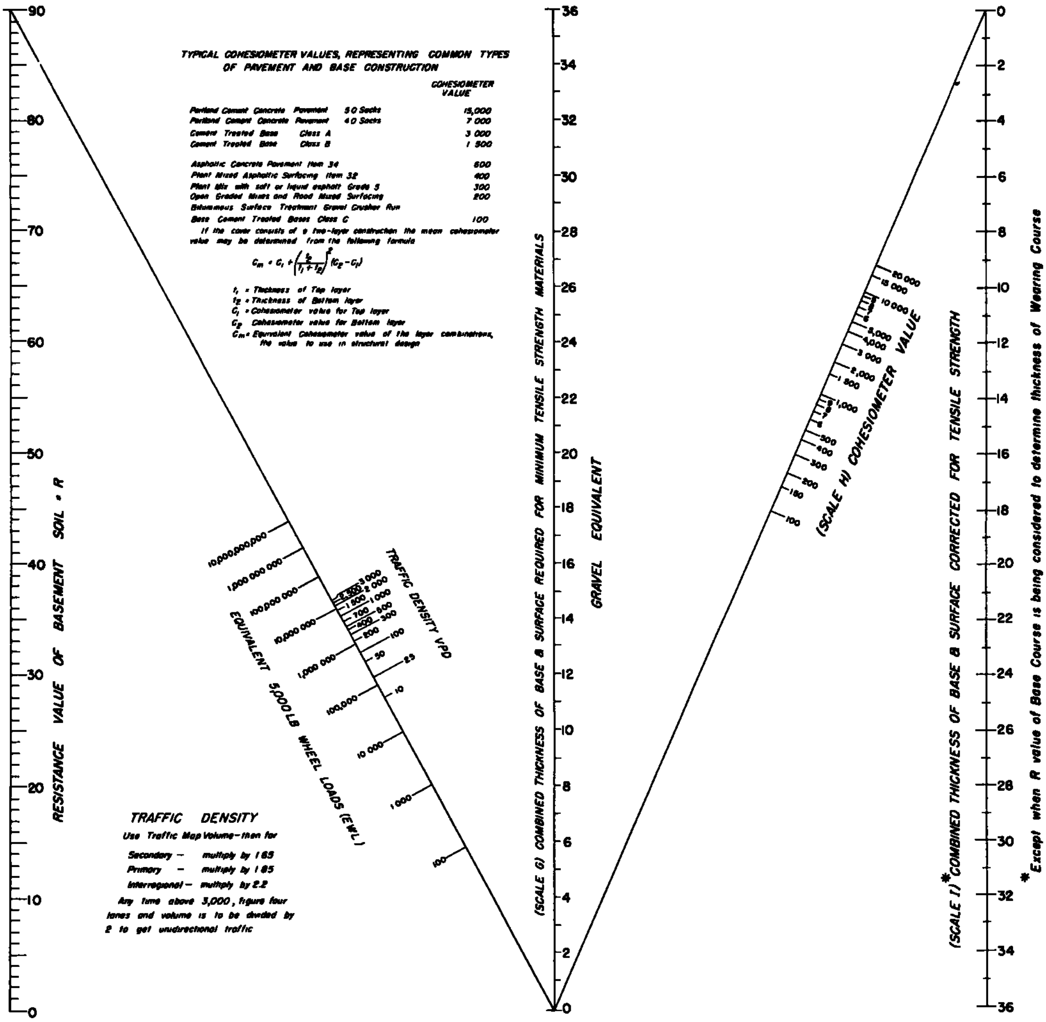


Figure 2. Thickness design chart for base and/or pavement.

TABLE 1

TABULATION OF DATA OF 11 SELECTED SITES BUILT USING THE DESIGN METHOD

Site No.	Project No.	Location	Year Built	Struct Sufficiency Rating[1] 1952	1954	Remarks
4	FI 44 (3)	Loveland Jct - Ft Collins Jct	1947	9-17-8	9-16-9	Generally good.
5	S 0024(1)	Sterling - West	1948	9-15-7	9-11-8	Foundation good, surface failing.
6	S 0009(1)	Sterling - East	1948	10-16-6	8-11-7	Foundation fair, surface failing.
9	F 193 (3)	Morrison - Conifer Jct	1948	8-13-8	4- 6-8	Unsatisfactory, sub-surface drainage failure.
10	F 292 (8)	Dowd - Wolcott	1948	10-19-9	9-16-8	Generally good.
11	F 019 (1)	Delta - West	1950	10-18-9	10-17-8	Generally good.
12	F 232A(1)	Grand Jct - Fruita	1948	10-17-9	10-17-9	Generally good, superior native material.
14	F 001-3(2)	Canon City - East	1949	10-19-9	10-19-9	Generally good.
16	S 0002 (3)	Springfield - Walsh	1948	9- 9-4	10-19-9	Foundation good, original surface treatment failed, new surface placed 1952.
22	F 006-1(1)	Adams City - North	1948	9-10-10	9-12-8	Foundation good, road mix surface failing under heavy truck traffic, no stability.
23	F 138-B & C	Muddy Pass - South	1948	10-20-10	8-16-9	Project at present is fair, original construction resurfaced in 1952.

[1] The sufficiency rating in Colorado awards a par of 40 points for structure. This is broken down to 10 points foundation, 20 points surface, and 10 points drainage. The figures given in the tabulation can be compared to these values.

to determine the adequacy of the design method. Shown in table form are the ratings for a number of projects designed and constructed under this method beginning in 1947. Ratings are shown terminating with those made in the spring and summer of 1955.

Table 1 indicates eleven projects that were built using the described design method. An examination of the table indicates two projects with unsatisfactory performance records. The first project at Site 9 and located between Morrison and Conifer, Colorado, failed because sub-surface drainage problems were not properly cared for at the time of construction. At Site 23, located near Muddy Pass, the failure has been adjudged to have been caused by an inferior foundation material.

The pit used on this project contained aggregate which was coated with a plastic material which had a definitely adverse effect when seasonal moisture permeated the foundation courses. This is one of the types of materials which cannot be properly evaluated by a simple sieve analysis in combination with Atterburg limits. The amount of material passing the No. 200 mesh is not sufficiently large to adversely affect the Atterburg limit test. When the same material is subjected to a stabilometer test its true potential is demonstrated. The change in the design method previously described will eliminate to a great extent the potential for this kind of failure occurring in the future.

All of the remaining nine sites built with the design method have good service records to date. There have been some demonstrated difficulties which have to do with wearing-course problems. As an example, in one place, a surface treatment was employed which did not have sufficient durability to withstand the type of traffic that uses the highway. In the other three cases, a road mix wearing-course was subjected to the type of beating that requires the stability of an asphaltic concrete. In none of the discussed cases was there any indication that the thickness determined by the design method was inadequate nor was there any deformation to indicate lack of stability in the foundation courses.

Reducing the tabular values to a description of the success or failure of the method, we find that 82 percent of the projects employing the method have been adjudged to have good performance and thus are classified as being successful. Unsatisfactory performance was exhibited in the remaining 18 percent and in the unsatisfactory areas, we believe that we now know the things that caused us to fail.

Table 2 shows the information on eleven sites of approximately the same era of construction as are represented by the other projects reported constructed under the principles of the design method. Eight of the eleven sites reported and that were not designed according to the described design method have either been reconditioned at the present time or, according to our own rating, should be rebuilt. The three successful areas are in the opinion of the writer located where the natural foundation soils are of a quality comparable to materials which would have been imported had a formal design method been used. Generally, the foundation soils on these three sites are of a sandy material which only needs to be confined in order to give it good bearing characteristic. Examining the good and bad performances on the eleven comparative sites, we find that the ratings would indicate that 27 percent of the projects have a satisfactory service record and 73 percent have an unsatisfactory record.

It is not to be judged that from this report that we have only built eleven sites to a

TABLE 2

A TABULATION OF DATA OF 11 SELECTED SITES BUILT JUST PRIOR TO THE ADOPTION OF THE FORMAL DESIGN METHOD

Site No.	Project No.	Location	Year Built	Struct Sufficiency Rating[1] 1952	1954	Remarks
1	FAP 150-D(3)	Elk Springs - Massadona	1947	6- 9-7	9-17-8	Original condition became untenable in 1952 and the project was rebuilt in 1953.
2	F 005-2(2)	Steamboat Springs - West	1949	7-11-8	8-16-8	Generally unsatisfactory. Extensive maintenance in 1954 and 1955, providing suitable sufficiency rating in 1955.
3	FAP 151-C(3)	Granby - Tabernash	1946	6- 8-8	10-20-10	The unsatisfactory condition which occurred in 1952 has been picked up by a construction project in 1955.
7	S 0111(1)	Holyoke - South	1948	8-13-3	7-11-6	Unsatisfactory, proposed for reconstruction.
8	F 040(3)	Brush - East	1947	10-17-7	9-16-7	Generally good.
15	F 006(7)	Lamar - South	1948	10-18-7	10-17-9	Present condition generally good. Extensive maintenance in 1949 for stabilization.
17	FI 002(15)	Trinidad -North	1948	10-18-7	10-18-8	Generally good.
18	S 0013(3)	Hooper - Moffat	1947	10-18-8	9-16-8	Generally good.
19	S 0122(2)	Del Norte - Northeast	1948	7-10-7	9-16-8	1952 condition required reconstruction in 1955.
20	F 298(11)	Pagosa Springs - East	1947	6- 3-7	7-13-8	1952 condition required extensive maint. in 1954. Present condition is only fair.
21	F 067(6)	Denver - West U. S. 6	1949	10-15-10	9-15-8	The 1952 condition had worsened in 1954 to require reconstruction in that year.

[1]The sufficiency rating in Colorado awards a par of 40 points for structure. This is broken down to 10 points foundation, 20 points surface, and 10 points drainage. The figures given in the tabulation can be compared to these values.

TABLE 3

GEOMETRIC DESIGN STANDARDS FOR RURAL HIGHWAY CONSTRUCTION, COLORADO DEPARTMENT OF HIGHWAYS

24 Hr Annual Avg Traffic, Design Period[a]	Pavement Type[b]	No. of Lanes	Lane Width (L)	Shoulder Width	Roadbed Width (R)	Design Speed	Maximum Grade %	R O W Width		Access Control (Desirable)	Max Curve Degree	Bridge and Separations		
								Desirable	Minimum			Design Load	Clear Width[g]	
													60' Long and Over	Less Than 60' Long
Type A				c				d	e					
5,000 - 15,000	High	4	12'										2L + 6'	R - 2'
1. Plains				10' - 4'	76' + Med	70	5	250'	150'	Full	3	H-20 S-16	Each	Each
2 Rolling				8' - 4'	72' + Med	60	6	250'	150'	"	5	" '	2	2
3 Rolling				8' - 4'	72' + Med	50	6	250'	150'	"	8 30'	" "	Lanes	Lanes
4 Mountainous				4' - 4'	64' + Med	40	6	250'	150'	"	14	' "		
Type B														
1,600 - 5,000	High	2	12'										2L + 6'	R - 2'
1 Plains				10'	44'	70	5	200'	120'	Partial	3	H-20 S-16		
2 Rolling				8'	40'	60	6	200'	120'	"	5	" "		
3. Rolling				8'	40'	50	6	200'	120'	"	8 30'	" "		
4 Mountainous				4'	32'	40	6	200'	120'	"	14	" "		
Type C														
800 - 2,000	High Med	2	11'										2L + 6'	R - 2'
1 Plains				8'	38'	70	6	150'	120'	No	3	H-20 S-16		
2 Rolling				8'	38'	60	6	150'	120'	"	5	' "		
3 Rolling				4'	30'	50	6	150'	120'	"	8 30'	' "		
4 Mountainous				4'	30'	40	6	150'	120'	"	14	' "		
Type D									f			h		
400 - 1,000	Medium	2	11'										2L + 6'	2L 6"
1. Plains				8'	38'	60	6	120'	80'	No	5	H-20		
2 Rolling				4'	30'	50	6	120'	80'	"	8 30'	"		
3 Mountainous				4'	30'	40	6[f]	120'	80'	"	14	"		
Type E									f			h	i	i
100 - 600	Low Med	2	10'										24'	24'
1. Plains				4'	28'	50	6	100'	60'	No	8 30'	H-20		
2 Rolling				4'	28'	40	6	100'	60'	"	14	"		
3 Mountainous				4'	28'	30	7[f]	100'	60'	"	24	"		
Type F									f			h	i	i
0 - 200	Low	2	-										24'	24'
1 Plains				-	26'	-	6	80'	60'	No	24	H-20		
2 Rolling				-	26'	-	8[f]	80'	60'	"	24	"		
3 Mountainous				-	22'	-	8[f]	80'	60'	"	24	"		

[a] The ' Types ' indicated refer to details shown on Department Standard M-4-F covering typical cross-sections The traffic volumes shown are based on annual average traffic volumes per 24 hours Since all designs are now based on the 30th highest hour, the following reference table is given for the purposes of correlation

Annual Average 24 Hr Volume	Equivalent 30th ø Highest Hour Traffic per Lane		
	P*	M*	T*
5,000	500	400	300
1,600	160	130	100
800	80	65	50
400	40	35	25
100	10	8	6

ø Unless actual traffic counts give a different value, the 30th highest hour is assumed to be 16 percent of the 24 hr annual average traffic volume

* AASHO classification for different types of traffic and used herein as follows

P = Predominantly passenger traffic = 0 to 10 percent trucks having wheel loads 5,000 lb and over.
M = Mixed traffic = 10 to 20 percent trucks having wheel loads 5,000 lb and over
T = Predominantly truck traffic = 20 percent trucks having wheel loads 5,000 lb and over

[b] High = portland cement concrete, asphaltic concrete, or equal.
High Medium = plant mix mats (2"+)
Medium = plant mix or road mix mats (2"+).
Low Medium = surface treatments and light road mix mats (2"-)
Low = natural gravel, sand clay, gravel or crushed rock
When comparative estimates indicate that a higher surface type can be constructed for a cost approaching the cost of a lower surface type, the higher type shall be used.

[c] In the case of divided highways, the larger dimension is the outside shoulder, the smaller the inside shoulder, when used.

[d] Desirable width is 200 feet with full access control, and where service roads are constructed outside of the right-of-way.

[e] Only with full access control, and where service roads are constructed outside of the right-of-way.

[f] In unusual cases the minimums shown may be altered after approval by the Denver Headquarters.

[g] For the interstate system, bridges less than 80 ft long shall have curb-to-curb width equal to the roadbed width, including shoulders and bridges 80 ft and over, the curb-to-curb width shall be the paved width + 6'

[h] Where the character of traffic is predominantly passenger vehicles or other unusual conditions exist, this loading may be reduced on order of the chief engineer

[i] Minimum bridge width on federal aid primary system shall not be less than 26'

preconceived design standard. Actually, at the inception of the design method in 1947, 26 areas were picked for evaluation. About one-half of the sites were currently under construction with the new design method and the others were sites that had been recently built which carried comparable traffic volumes and which were located in environmental conditions similar to the new areas of construction that had been selected. Out of the 26 original sites, 22 are still available for examination and they have become the basis of information which has here been presented. The other 4 sites have either been transferred off the state system, or have been rebuilt for reasons other than structural failure.

Figure 1, which was previously used in exhibiting the design curve information, can be used to determine the over-all thickness of pavements on the various classifications of soils. The design curves indicate that for foundations which approximate surfacing values, a minimum of 4 in. of a pavement structure would be employed. Actually, the 4-in. thickness is nominal and has been established as the minimum that can be used with out present construction equipment to lay a uniform base and wearing course. At the other extreme, under the lightest type traffic and under adverse soil, frost and saturation conditions, a pavement of 17 in. would be employed. For the same conditions and under the heaviest traffic, the pavement structure would be a minimum of 10 in. thicker or would approximate 27 in. in total thickness. Moderate traffic would about split the difference and the pavement thickness would approximate 22 in. The author has always had a personal antipathy for the use of the terms, light traffic, medium traffic, and heavy traffic. It is believed that these generalized terms should not be used in the presumptions used in designing a pavement structure. A heavy duty road on a transcontinental route in the far reaches of the western United States might carry as little as 3,000 to 4,000 vehicles a day. This would be referred to as heavy traffic whereas the same volume in proximity to one of the big metropolitan centers of the east would be considered to be very light. Actually, the traffic volume must be considered, not only in numbers but should be related to wheel loads and repetitions of those loads by magnitude. The performance related to that type of criterion, which would have significance, does not attach itself to the generalized terms spoken of above.

In conjunction with the design method, we may use road mix bituminous pavements as light as 1½ in. for traffic volumes of less than 500 vehicles per day. For the volumes between 500 and 1,500, a hot mix of not less than 2 in. is most commonly used. Above that volume, 3 in. of hot mix is employed and is usually placed in two 1½-in. layers. Any of these basic thicknesses and types may be varied for unusual conditions such as abnormal volumes of heavy truck loads, non-availability of desired grades of aggregate or other similar conditions.

Resurfacing and reconstruction projects are handled in a manner identical to new construction, that is, the same soil survey and analysis of materials are made and the design proceeds in a perfectly normal manner as if the road were to be newly constructed. The criteria for the selection of surface types and the change from one traffic category to another are predetermined on the basis of a set of standards which are a part of the department's field and office manual. Shown as Table 3 are the design criteria which are employed by the department.

Discussion

RAYMOND C. HERNER, Chief, Airport Division, Technical Development and Evaluation Center, Civil Aeronautics Administration, Indianapolis — Livingston's paper is of real value because it describes the service records of roads which have been in use long enough to give some indication of their ultimate behavior. In a refreshingly frank manner he enumerates his failures as well as his successes but does pause long enough to indicate some plausible reasons - outside the realm of the designer - which may account for the failures. This is a point much too frequently overlooked. It is common practice to assign all failures to inadequate design, whereas they often are caused by poor construction practices or control.

Design of Flexible Pavements in Alabama

J. L. LAND, Chief, Bureau of Materials and Tests
Alabama State Highway Department

We shall attempt to show how the development of soil and material evaluation has given our engineers criteria for designing and predicting the service value of a flexible pavement over any of the ordinary soils encountered throughout the state.

The paper will concern itself with subgrade soil evaluation, materials evlauation (giving methods employed), use of soils and materials in sequence of the supporting value, and surface types; the design employed on two or more road projects over different soil types and employing different base materials; and a brief resume of service behavior of these projects.

We shall attempt to abstain from entering into highly controversial subjects, such as special methods or apparatus.

● THE demands for all-weather surfaced roads in the early 1930's and the shortage of construction funds with which to build them forced Alabama into a program of low-cost highways.

Some very bad experiences were encountered in the beginning, but were soon overcome and resulted in soil evaluation, the use of local materials (soil and soil aggregates), and the development of an extensive yearly construction program of flexible pavement.

The development of this program somewhat revolutionized previous methods of construction, in the use of soils and materials. Evaluation for each of these items was begun and service value established for design criteria based on calculated items, just as was done for all other structures.

In the development of the low-cost road program as mentioned above, studies of soil, use of local materials, drainage and compaction, demonstrated that economical and satisfactory roads could be built in most sections of the state from local products, and that a high class structure could be built commensurate with current traffic needs, from flexible pavement.

This program of design has been revised upward to keep pace with increased loads, load frequency, and increased tire pressure, affecting both foundation courses and pavement.

Our idea of flexible pavement design primarily concerns itself with an ultimate smooth, even, resiliant and non-skid textured surface for use of wheeled equipment, predominately equipped with pneumatic tires. When properly designed, it absorbs load stresses and transmits them to the subgrade or original supporting soil in the calculated increments which it is capable of supporting, without distortion or ruptures.

The elements that enter the problem of accomplishing the end point of design are many, with the most necessary being drainage, subgrade soil, foundation courses, and selection of surfacing or pavement, and their being properly controlled and utilized through construction efforts.

Our design of flexible pavement is actually a method of evaluating subgrade soils, providing proper types or types of foundation media, placed in zones or strata in sequence of their increased supporting value from subgrade to pavement, and providing a suitable wearing course or courses.

We have studied and tried in a limited way most all methods of evaluation, with the conclusion that the best to employ are those that are reproducible and can be employed in an economical and satisfactory manner. The simpler and easier the method, the better the inspection and construction.

Our current method employed for determining the working value of soils and soil aggregates are soil analysis, BPR soil classification and group index curves, supplemented by modified California Bearing Ratio Test (2000-lb static load, which most nearly conforms to specification requirements, based upon the use of currently available construction equipment). See Figures 1 and 2.

Preliminary studies of soil occurring in the subgrade, materials available to the

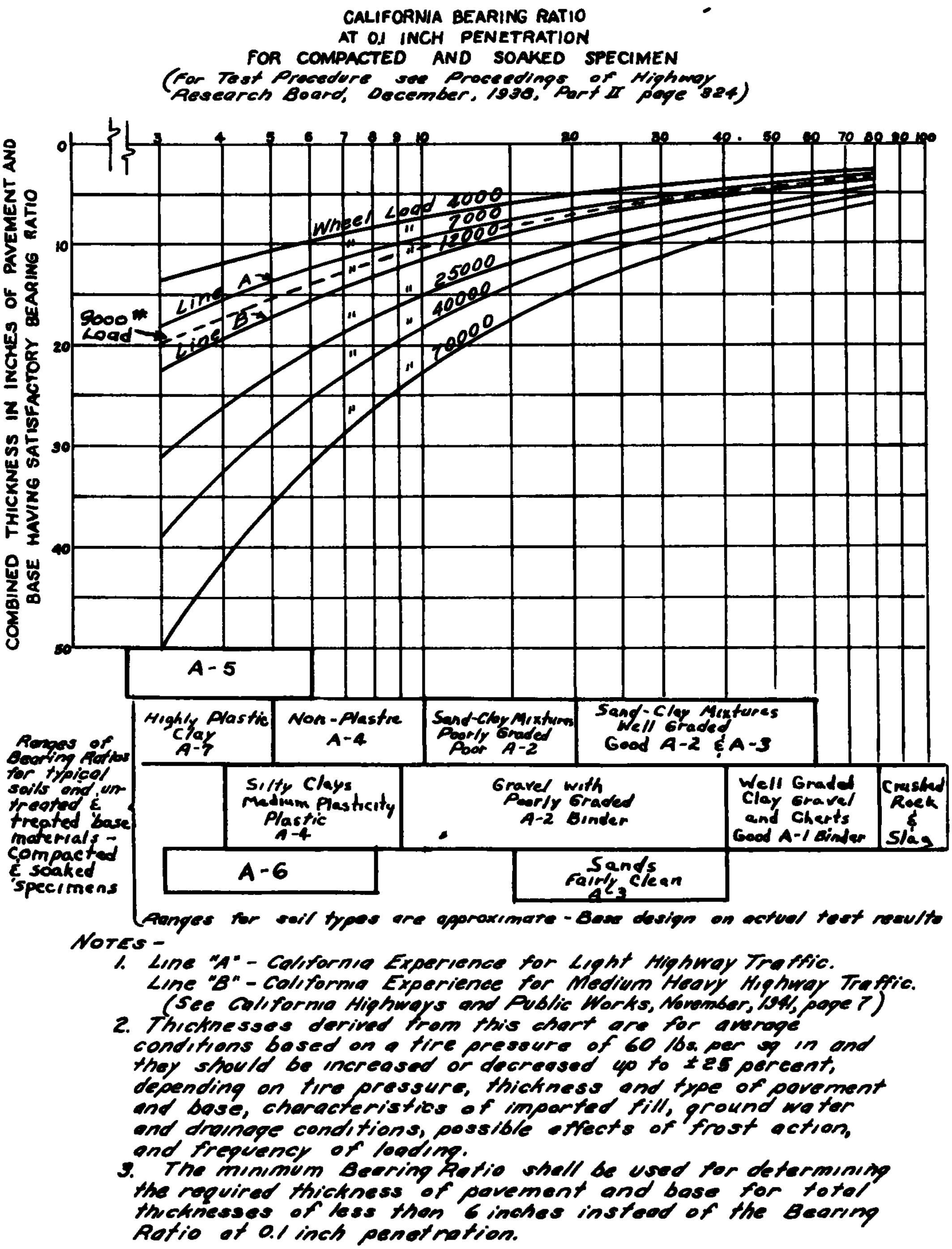


Figure 1. Tentative design of foundations for flexible pavements.

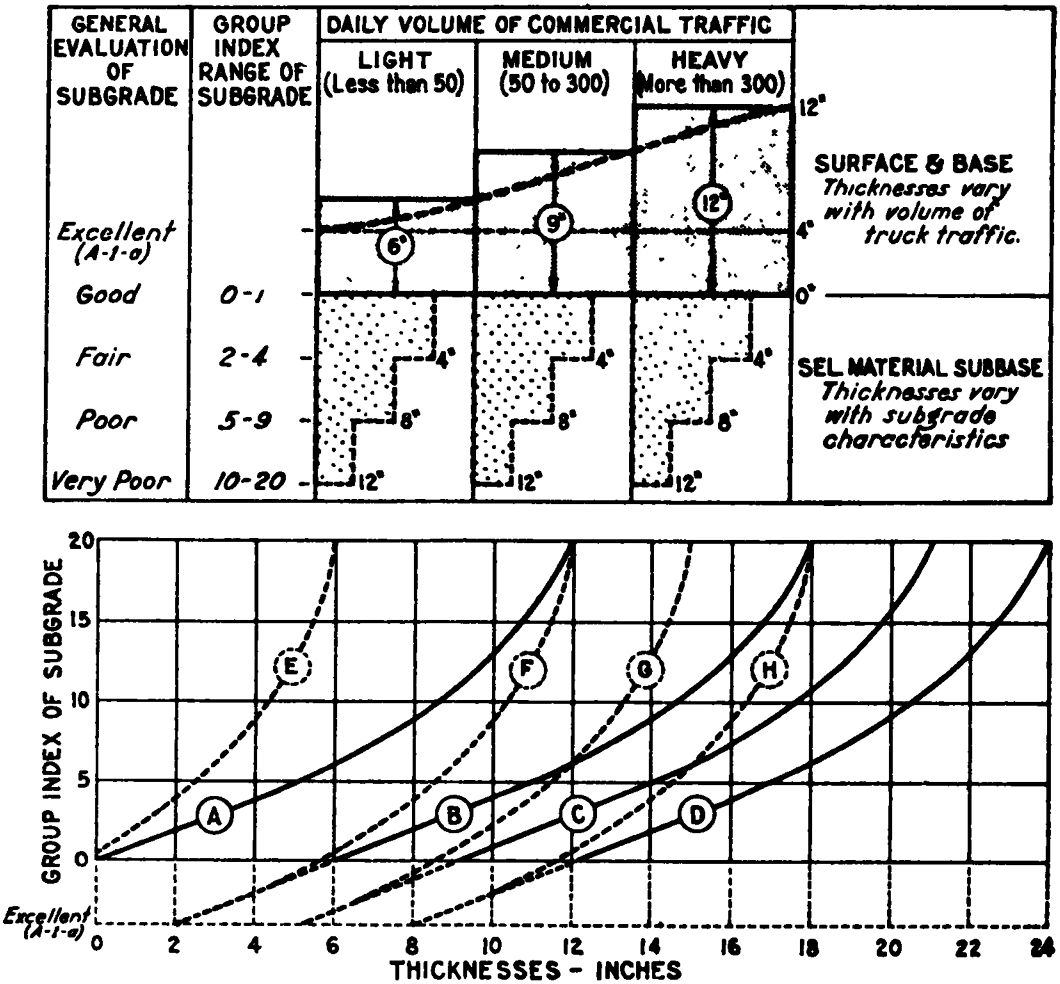
project and traffic demands determine the foundation media and the surfacing type, and require:

1. Soil Survey - See Appendix A for method.
2. Materials Survey - See Appendix B for method.
3. Traffic Count.

4. Evaluating Subgrade Soils.
5. Evaluating Materials Available.
6. Surfacing - See Appendix C for method.

Our Planning Survey Division supplies us with a traffic flow map for each proposed project, giving the number and type of vehicles using the road, an estimate of future increases to be expected due to improvement, new feeder lines, and for expected increase in loads.

We evaluate both soil and foundation material by the same method because these range



Curve A.- Thicknesses of Selected Material Subbase only.

" B.- Combined Thicknesses of Surface, Base & Selected Material Subbase - Light Traffic

" C.- " " " " " " " " " Medium

" D.- " " " " " " " " " Heavy

" E.- Thicknesses of Additional Base which may be substituted for Subbase of Curve A.

" F.- Combined Thicknesses of Surface and Base (No Subbase) - Light Traffic.

" G. " " " " " " " " Medium "

" H. " " " " " " " " Heavy "

Figure 2.

from a soil of the worst degree to aggregates with excellent soil binder of filler.

Preference has been given to inert soils for subgrade and aggregate binders since the beginning, but actual measurement of their working values by test was not begun until the late 1930's when the writer visited the California Highway Department and became impressed with their CBR device for measuring shear. We started actual use of CBR values, or relative values derived therefrom, in 1939 in conjunction with the old Bureau of Public Roads classification and continued the same until the new HRB classification and group index curves were adopted. We immediately began using group curves as a supplement to CBR values. This action enabled us to place comparative working knowledge of soils into the hands of more engineers and inspectors than the use of CBR values alone.

It was realized in the beginning that design within itself was of little value unless fully developed on construction. Consequently, every effort was made to simplify or make easy the design criteria, so that better understanding and cooperation could be had alike from inspectors and contractors; hence, joint use of BPR and CBR curves.

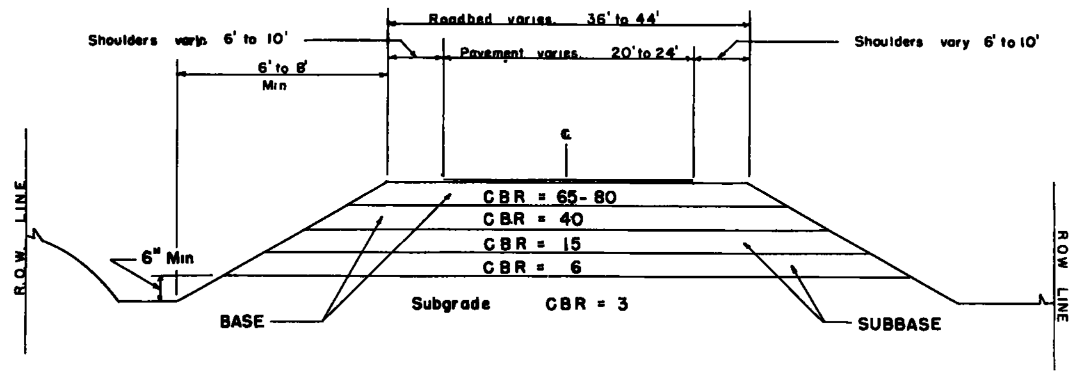
To date, little or no condusion has arisen and much common knowledge has resulted from our procedure of operations.

We employ triaxial apparatus and do some check work by that method. However, the most of our triaxial values are on undisturbed samples, or remolded specimen at a specified degree of density and moisture for a definite measurement, and this is generally for foundation determination of soil in situ.

We pursue the most economical and feasible method of design for foundation courses, beginning with the subgrade soil and constructing subsequent layers to yield satisfactory value in sequence of its position in the roadbed and resulting in material of satisfactorily high bearing value immediately below the pavement structure.

Armed with complete information as to the type and supporting value (based on specification requirements) of the soil in the subgrade on all parts of the project length, an inventory and evaluation of local and manufactured materials available, and a traffic count with estimated service demands, we proceed with design. We are not concerned with freezes of more than 8 in. deep.

With the assembled data in hand, we make a field reconnaissance and set up foundation thickness of layer type for the entire length of the project, for design and estimating purposes, varying the total depth as required to yield a uniform carrying value for its entire extent. The actual depth of foundation is based upon soils occurring in subgrade after grading and drainage are completed. See Figures 3 and 7.



Subgrade with CBR = 3
Total thickness would be 20" for 12,000 pound wheel load
Employ 6" CBR = 6
Employ 6" CBR = 15
" 4" CBR = 40
" 4" CBR = 65-80
" 1½" Surfacing

Figure 3. Typical section.

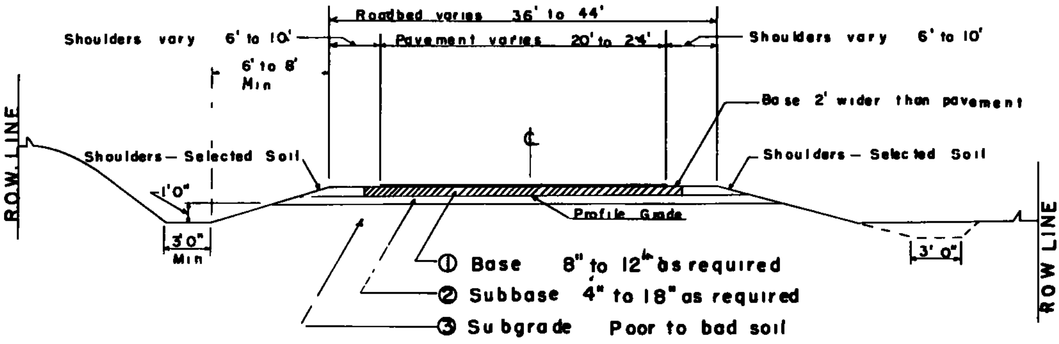


Figure 4. Typical section, selected soil shoulders.

Should the surface type be an armor course of less than 2 in., no consideration is given for its value in the structure. Should it be of high type 4, 5, or 6 in. thick, the total depth less 2 in. is deducted from foundation thickness.

When practical, we employ controlled grading, but if this is not feasible, we normally employ selected roadside materials (the best and most economical) for the lower layers of foundation courses (subbase), with each subsequent layer (from the subgrade up) having the proper soil constants and bearing value in sequence of its position, always reserving a requisite depth of high bearing value material suitable for the intended purpose of a finished base. Our bases range 4 in., 6 in., 8 in., and 10 in., as required to meet traffic demands, (farm-to-market, secondary or primary roads) with the subbase varying in depth to compensate for varying soils, thereby providing a uniform bearing value throughout the length of the project.

There is a tendency for some engineers to employ materials of higher bearing values and use less depth for foundation courses instead of the layer type design. Our experience indicates that aside from the economics involved, in so doing, one is drifting towards a rigid pavement design rather than pursuing the mechanics of a flexible structure. Consequently, we stick to the layer method when at all possible and this is 100 percent to date.

The top 4 in., 6 in., 8 in., or 10 in., as the case demands, of foundation course, designated as base, is of a carefully selected material and ranges from a high-grade A-2 soil to a crushed stone sand-filled soil concrete, and includes sand-clay, topsoil, sand-clay gravel, sand-filled cherts and sand-filled crushed aggregate (slag or stone). Their CBR values range approximately from 60 to 150 standard to 35 to 150 soaked.

When sand-clays and topsoils are employed as base (we have some 4,000 miles of this type, some carrying 5,000 vehicles of mixed traffic daily), roadbeds are built high and well drained, with shoulders of ¾ in. slope to the foot to keep moisture content low and maintain the highest shear value possible (Standard CBR) in the base materials, during adverse weather. This is our weak base type and must have pervious shoulders. See Figures 4 and 5.

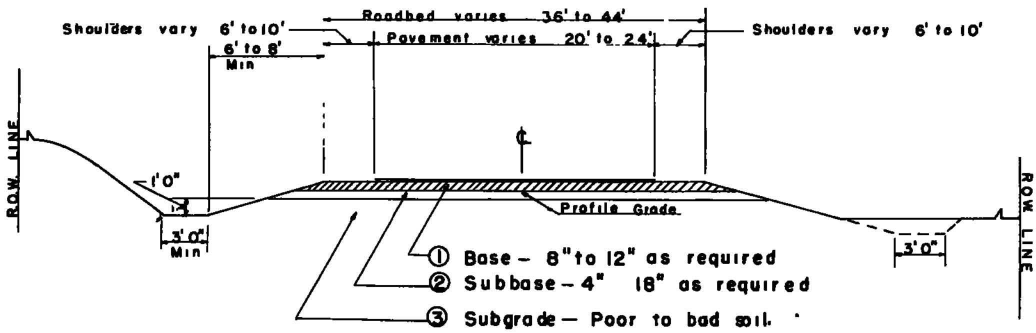


Figure 5. Typical section, base full width.

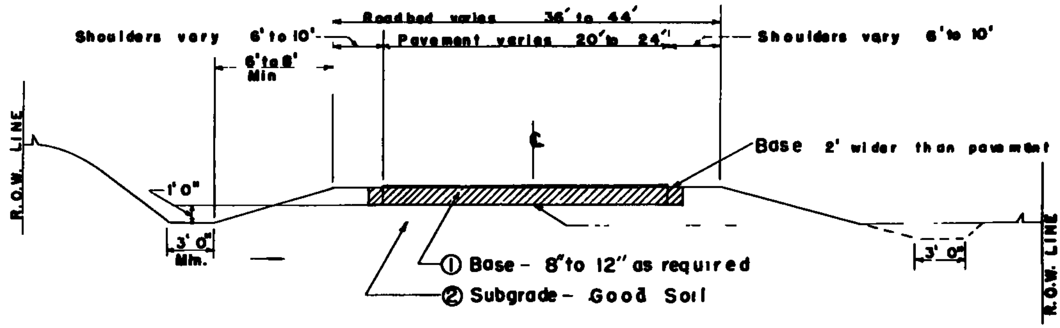


Figure 6. Typical section, trench section.

The poorer grade sand-clay gravels, poor class of chert, or either of these materials with poor binders requiring the same type of roadbed and drainage as sand-clays, are termed medium bases, and require pervious shoulders.

The better bases are sand-filled chert, sand-clay gravels, with excellent binder or soil aggregate concrete made from crushed aggregate and reasonably clean sand. They yield feasible construction, high shear value with a small loss when saturated. These are the more expensive and stronger bases, and can be employed anywhere with success, provided a suitable subgrade is prepared as required. See Figure 6.

When we discuss surfacing or pavement proper, we want it fully understood that: (1) the subgrade soil has been fully evaluated at the degree of density specified for the entire length of the project; (2) the subbase has been built of the materials, both as to quantity and quality at the optimum density, and to the section specified; and (3) the base course has been built of the materials specified as to quality, quantity, optimum density

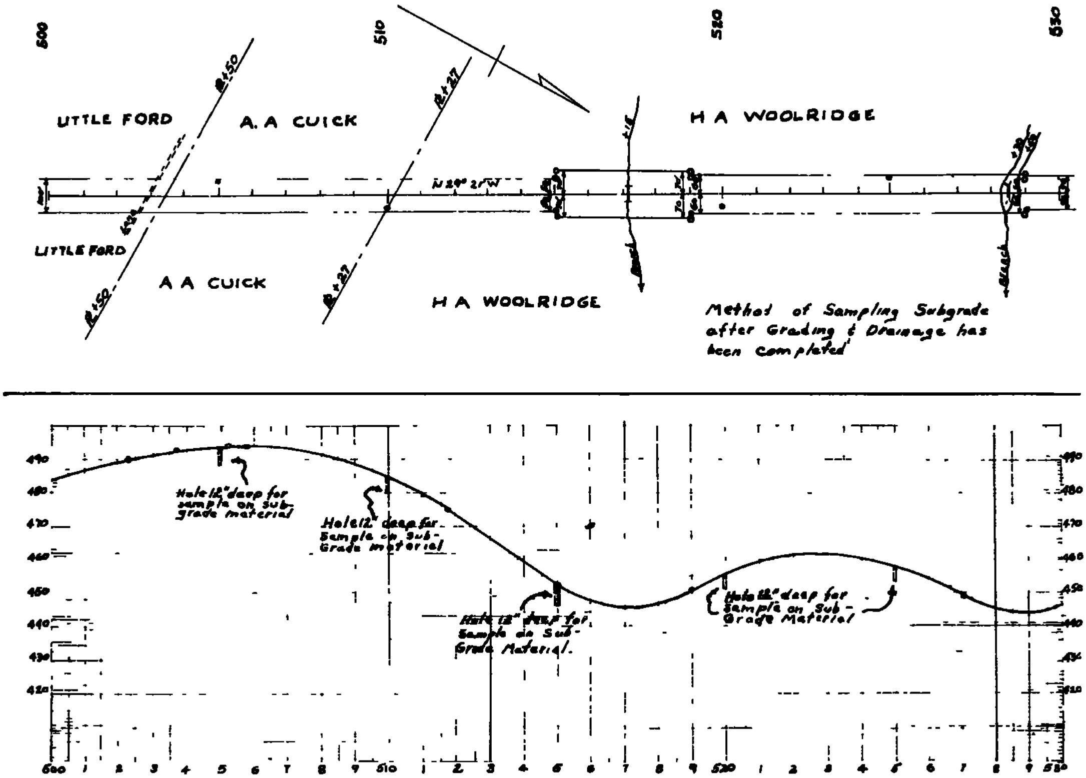


Figure 7.

and section and is ready for the surfacing.

The public demands a smooth, easy-riding and non-skid surface and we make every effort to secure this effect. Some of our best results in obtaining a good riding roadway surface have come from stage construction with frequent applications of leveling and light plant mix surface courses of suitable gradation and bitumen content ($^3/_4$ in. application every 4 to 7 years). They range from surface treatment or armor courses (1 in. to 2 in. thick) to 4 in. or more in depth, placed in two or more courses.

Surface treatments are aggregate courses placed inverted penetration type on a primed base course. The first course is made from fairly coarse aggregate as near uniform size as can be readily obtained, choked with a second course of smaller particles and covered with approximately $^3/_4$ in. of fine graded plant-mix. However, we sometimes place 1 in. of plant-mix directly on a primed base.

Someone may question: "Why a prime or why the Keystone Course of aggregates?" Both have a very definite function in our line of reasoning since we have hot climate, heavy rains and employ weak bases. The prime serves to seal out moisture or prevent rapid evaporation and as a dowel to tie the pavement to the base, while the hot application supplements the prime in doing its work and when covered with aggregate provides increased stability and a rough textured surface to hold the plant-mix.

When building the higher type of pavement, the aggregate course is usually eliminated, but the prime is retained for reasons given above. Such pavements are usually divided into bottom and top courses in such a ratio as to facilitate the use of largest aggregates possible in the mix of both layers and to enable us to secure the required density from a reasonable amount of rolling, thereby promoting economy, and service value commensurate with expended effort.

Our standards of design for plant-mix are the surface area method for determining bitumen content, sieve gradation of aggregates for uniformity, and density and stability for service life.

We employ preliminary samples of materials and make pilot mixes for design. Then plant inspection for control on all plants making mixes, and roadway inspection for placement. The bitumen content is checked by extraction, the aggregate by screens, the density by specific gravity method, and stability by Bruce Marshall apparatus, or other standard means as specified.

We attempt to employ the simplest standard method that is reproducible for the specific test, then establish relative values for that obtained by other methods when possible. This line of reasoning assists us in placing workable or understandable knowledge in the hands of the people, who actually do the work, thereby serving as an incentive to build the design into the structure.

A condensed description and table of design data for three typical projects are listed below, giving the main details of design employed — U. 352, F. 286 (1), and F. 120 (3).

Project U. 352, Montgomery County, is of four-lane depressed parkway construction, extends over conditions which approach the lower limits of the bad soils occurring through the nation. It was carried through the winter of 1954-55 on construction and gives every indication of being a satisfactory highway.

Project F. 286 (1), Madison County, is of four-lane depressed parkway construction, right through the center of the City of Huntsville, with a low grade line, requiring increased safety factors.

It is now completed and gives every indication of being a most excellent thoroughfare.

Project F. 120 (3), Bibb County, is on a federal primary route and will be subjected to heavy truck traffic, with quite a prospective increase. It is of two-lane construction, having a 42 ft roadbed, and 24 ft pavement width respectively. The terrain it traverses is of a rough hilly nature, with stratified soils, ranging from good to poor; hence, the three types of soil occurring in the subgrade. The roadbed is high with the bottom course of the base of sandy material and very pervious. Past experience with soil types and materials employed indicate very satisfactory road service.

TABLE 1

SUMMARY OF DESIGN

Project: U. 352 (1) Montgomery Co
Total cost per sq yd for subbase, base and surfacing - $2.33
Traffic: 7,800 V/D 20 percent commercial 1965 estimate
Rainfall: 40 in. to 60 in. a year
Depth of Freeze: 2 in.
Terrain: Rolling coastal plain

Station	Subgrade Classification	Subgrade CBR	Subbase Type	Subbase CBR	Base Type	Base CBR	Pavement Type
42+00	A A	7.7	Bottom 5" sand-clay	34.5	10" clay gravel		Double Bit. S. T.
	A-4 (3)		Top 5" sand-clay	46.0		60.0	80lb/SY Plant Mix
120+00	A-2		Bottom 5" sand-clay	34.5			Double Bit. S. T.
	A-4 (2)	14.9	Top 5" sand-clay	46.0	10" clay gravel	60.0	80lb/SY Plant Mix
184+00	A-7		Bottom 5" sand-clay	34.5			Double Bit. S. T.
	A-7-6 (15)	7.8	Top 5" sand-clay	46.0	10" clay gravel	60.0	80lb/SY Plant Mix
258+00	A-7						Double Bit. S. T.
	A-7-5 (20)	3.2	10" clay gravel	47.0	10" clay gravel	59.1	80lb/SY Plant Mix
287+00	A-7						Double Bit. S. T.
	A-7-6 (12)	5.0	10" clay gravel	47.0	10" clay gravel	59.1	80lb/SY Plant Mix

Remarks: Clay gravel used as base is on sandy side. Clays in subgrade are organic type. Pavement thickness 2 in. Top 6 in. of subgrade, subbase and base compacted to 100 percent density method T-99-42. Two in. of pavement used initially to be followed in three years with a leveling course and 1½ in. of wearing course.

ANALYSIS OF MATERIALS

	Subgrade sta 42+00	Subgrade sta 120+00	Subgrade sta 184+00	Subgrade sta 258+00	Subgrade sta 287+00	Bottom layer subbase 42+00 to 184+00	Top layer subbase 42+00 to 184+00	Subbase 258+00 to 287+00	Base 42+00 to 184+00	Base 258+00 to 287+00
Total % Passing										
2"									100.0	100.0
1"							100.0	100.0	98.0	92.0
No. 4				100.0			92.0	73.5	75.0	69.0
No. 10	100.0	100.0	100.0	99.0	100.0	100.0	84.0	64.6	55.0	65.0
Matl Pass. 10 M %										
Clay	Not	Not	Not	78.6	55.2	12.4	15.0	24.2	14.0	16.0
Silt	Run	Run	Run	15.3	28.7	14.6	8.9	3.2	10.6	7.5
40	99.7	96.6	98.7	99.1	97.0	98.3	67.5	67.7	58.7	55.3
60	98.2	90.0	97.6	97.8	93.3	84.4	43.1	45.6	41.0	32.4
200	51.2	44.1	87.2	93.9	83.9	27.0	23.9	27.4	24.6	23.5
F. M.	26.3	19.4	32.0	53.5	33.8	15.8	21.2	26.3	21.5	21.5
L. L.	29.8	21.4	47.3	65.9	41.0	19.4	26.4	27.8	28.8	28.8
P. L.	19.4	16.9	23.7	45.8	23.0	15.6	19.4	20.9	19.4	21.4
P. I.	10.4	9.5	23.6	20.1	18.0	3.8	7.0	6.9	9.4	7.4
S. L.	18.5	17.4	20.1	23.5	18.3	17.4	22.1	22.0	22.7	21.6
V. C.	13.5	3.4	19.5	47.1	27.9	0.0	0.0	7.1	0.0	0.0
L. S.	4.13	1.11	5.77	12.19	7.87	None	None	2.26	None	None
S. R.	1.73	1.72	1.64	1.59	1.80	1.78	1.68	1.75	1.72	1.70

TABLE 2

SUMMARY OF DESIGN

Project: F. 286 (1) Madison Co
Total cost per sq yd for subbase, base, and surfacing - $2.13
Traffic: 5,000 V/D 15 percent commercial 1965 estimate
Rainfall: 40 in. to 60 in. a year
Depth of Freeze: 8 in.
Terrain: Rolling river valley

Station	Subgrade Classification	Subgrade CBR	Subbase Type	Subbase CBR	Base Type	Base CBR	Pavement Type
27+00	A-7 A-7-6 (10)	12.0	6" sand-clay	17.3	10" crushed limestone with sand-clay binder	137.8	Double Bit. S.T. 80lb/SY Plant Mix
57+00	A-7 A-6 (9)	13.2	6" sand-clay	17.3	10" crushed limestone with sand-clay binder	137.8	Double Bit. S.T. 80lb/SY Plant Mix
92+00	A-4 A-6 (5)	11.0	6" sand-clay	17.3	10" crushed limestone with sand-clay binder	137.8	Double Bit. S.T. 80lb/SY Plant Mix
152+00	A-4 A-6 (9)	13.8	6" sand-clay	17.3	10" crushed limestone with sand-clay binder	137.8	Double Bit. S.T. 80lb/SY Plant Mix
200+00	A-7 A-7-5 (9)	12.7	6" sand-clay	17.3	10" crushed limestone with sand-clay binder	137.8	Double Bit. S.T. 80lb/SY Plant Mix

Remarks: High subgrade CBR values in soil are due to cherty material retained on No. 10 screen. Total pavement thickness 2 in. Clays in subgrade are inorganic type. Top 6 in. of subgrade, subbase and base compacted to 100 percent density method T-99-42. Two in. of pavement used initially to be followed in 3 years with a leveling course and 1½ in. of wearing course.

ANALYSIS OF MATERIALS

	Subgrade Sta 27+00	Subgrade Sta 57+00	Subgrade Sta 92+00	Subgrade Sta 152+00	Subgrade Sta 200+00	Subbase 0+00 to 200+00	Base 0+00 to 200+00
Total Passing %							
2"							100.0
1"	100.0		100.0				95.0
No. 4	90.7	100.0	86.1	100.0	100		37.8
No. 10	81.6	99.0	66.6	99.0	99.0	100.0	32.6
Matl Pass. 10 M %							
Clay	66.6	57.0	58.4	63.0	56.6	12.0	15.0
Silt	19.0	27.3	25.6	29.0	22.4	6.8	10.2
Total Sand	14.4	15.7	16.0	8.0	21.0	81.2	74.8
40	97.0	96.3	96.5	99.2	97.6	85.8	81.6
60	94.9	93.6	94.0	98.2	95.0	40.7	50.0
200	85.6	84.3	84.0	92.0	79.0	18.8	25.2
Field Moisture	35.5	36.5	26.9	26.4	35.2	14.3	17.0
Liquid Limit	44.4	39.2	35.9	35.2	42.0	17.1	16.9
Plastic Limit	29.0	27.1	23.9	23.0	31.4	12.9	17.4
Plasticity Index	15.4	12.1	12.0	12.2	10.6	4.2	0.0
Shrinkage Limit	18.9	24.3	18.5	17.9	23.9	13.8	15.0
Volume Change	28.4	19.9	15.0	15.2	18.5	0.9	3.7
Lineal Shrinkage	8.00	5.87	4.55	4.61	5.50	0.29	1.20
Shrinkage Ratio	1.71	1.63	1.79	1.79	1.64	1.83	1.83

TABLE 3

SUMMARY OF DESIGN

Project: F. 120 (3) Bibb County
Total cost per sq yd for subbase, base and surfacing - $1.38
Traffic: 1,600 V/D 35 percent commercial 1965 estimate
Rainfall: 40 in. to 60 in. a year
Depth of Freeze: 3 in.
Terrain: Rolling foothills

Station	Subgrade Classification	Subgrade CBR	Subbase Type	Subbase CBR	Base Type	Base CBR	Pavement Type
230+00	A-2				Bottom 5" layer sand-clay	34.0	Double Bit. S.T.
	A-4 (0)	28.3	None		Top 6" 70% chert 30% sand	88.7	80lb/SY Plant Mix
290+00	A-4				Bottom 5" layer sand-clay	21.5	Double Bit. S.T.
	A-6 (10)	7.3	6" sand-clay	32.6	Top 6" 70% chert 30% sand	88.7	80lb/SY Plant Mix
375+00	A-4				Bottom 5" layer sand-clay	21.5	Double Bit. S.T.
	A-4 (8)	8.3	4" sand-clay	30.7	Top 6" 70% chert 30% sand	88.7	80lb/SY Plant Mix
575+00	A-4				Bottom 5" layer sand-clay	21.5	Double Bit. S.T.
	A-6 (8)	7.0	6" sand-clay	34.1	Top 6" 70% chert 30% sand	88.7	80lb/SY Plant Mix

Remarks: Bottom layer base used from Sta 290+00 to 575+00 is sandy. Top 6 in. of subgrade, subbase and base compacted to 100 percent density method T-99-42. Two in. of pavement used initially to be followed in 3 years with a leveling course and 1½ in. of wearing course.

ANALYSIS OF MATERIALS

	Sub-grade Sta 230+00	Sub-grade Sta 290+00	Sub-grade Sta 375+00	Sub-grade Sta 575+00	Sub-base Sta 290+00	Sub-base Sta 375+00	Sub-base Sta 575+00	Bot layer base 230+00	Bot layer base 290+00 575+00	Top layer base 230+00 575+00
Total Passing %										
2"										100.0
1"										88.3
No. 4					100.0	100.0		100.0	100.0	54.1
No. 10	100.0	100.0	100.0	100.0	99.0	99.0	100.0	99.0	99.0	43.9
Matl Pass. 10 M %										
Clay	20.0	Not	26.0	30.0	18.6	24.6	16.4	14.6	14.6	15.2
Silt	19.9	Run	52.4	46.0	13.5	6.2	5.0	3.3	6.3	12.0
Total Sand	60.1	7.2	21.6	24.0	67.9	69.2	78.6	82.1	79.3	72.8
40	91.6	97.9	99.6	98.9	85.8	77.2	81.5	87.3	70.0	77.8
60	71.0	95.3	98.7	98.0	69.1	58.1	48.4	59.0	38.2	50.7
200	39.9	92.8	78.4	76.0	32.1	30.8	21.4	17.9	20.7	27.2
F. M	15.0	25.2	26.0	25.8	15.7	20.8	18.9	20.4	14.7	17.1
L. L	14.6	37.8	31.6	34.7	17.8	25.2	22.8	21.8	17.0	19.7
P. L.	14.3	23.0	24.1	24.1	15.6	17.1	17.9	19.5	14.8	16.7
P. I.	0.3	14.8	7.5	10.6	2.2	8.1	4.9	2.3	2.2	3.0
S. L.	13.5	18.6	24.8	20.0	14.6	18.0	17.8	23.2	16.7	15.6
V C.	2.8	11.4	1.9	9.9	2.0	5.0	1.94	0.0	0.0	2.74
L. S.	0.92	3.54	0.65	3.10	0.66	1.62	0.65	None	None	0.89
S R.	1.85	1.73	1.56	1.70	1.82	1.79	1.77	1.63	1.79	1.83

Appendix A

Soil Surveys

The evaluation of subgrade soil requires surveys which provide the sampling, analyzing and designating soil type and value for each 100 to 1,000 feet of roadway length, and establishing a soil profile from which the required thickness of foundation media is determined.

Purpose of Soil Survey

1. To predetermine by analysis of soil samples taken from the prototype, the type or types of soil that will be encountered in the construction of a proposed highway project.

2. To determine from (1) above the working value of the soil that will occur in the subgrade of a highway project on each section of roadway throughout its entire length.

3. To determine from (1) and (2) above the disposition of soil on construction in such a manner as to yield the best final results and longest road service value.

4. To determine from (1), (2), and (3) above the finished roadway subgrade needs for treatment, the quality and amounts of subgrade reinforcement (in addition to base course or pavement) that will be required for the roadway surfacing to support adequately the intended traffic loads, during all variations of weather and moisture.

5. To analyze the findings of operations (1), (2), (3), and (4) above and compile a report for use in making recommendations for the design of the project affected.

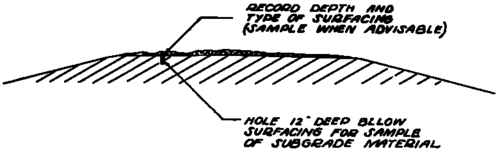


Figure A.

Scope

1. Soil surveys cover the examination of the soil immediately within the limits of proposed construction, both as to width and depth, and, to areas adjacent to the roadway section so far as is necessary to obtain complete soil data for use in design.

2. They extend to the investigation of the subgrade for each section of the entire project length whether cut or fill and furnish information as to the expected conditions (that will be encountered on construction) of the subgrade in cuts and foundation soil over which the fills are to be made. When an old alignment and grade is closely followed and surfacing has been employed, this investigation shall include both depth and type of surfacing and subgrade soils (see Figure A).

3. They extend to the examination of road building material when advisable and to

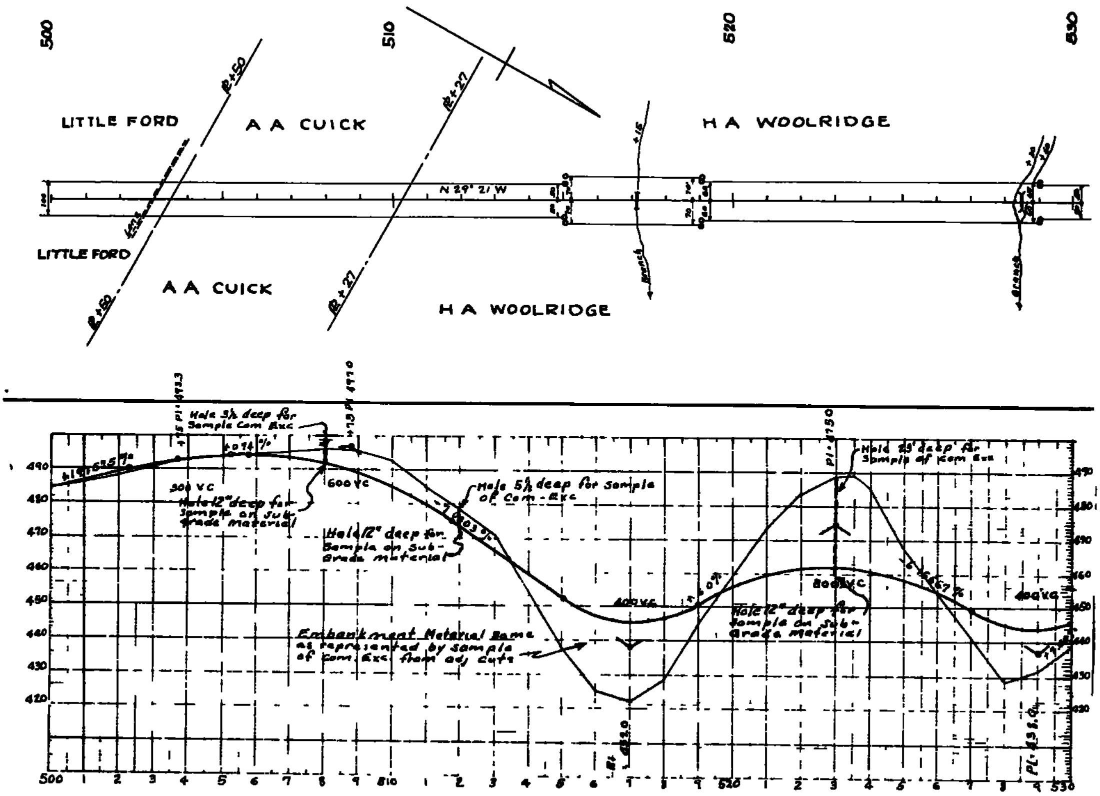


Figure B.

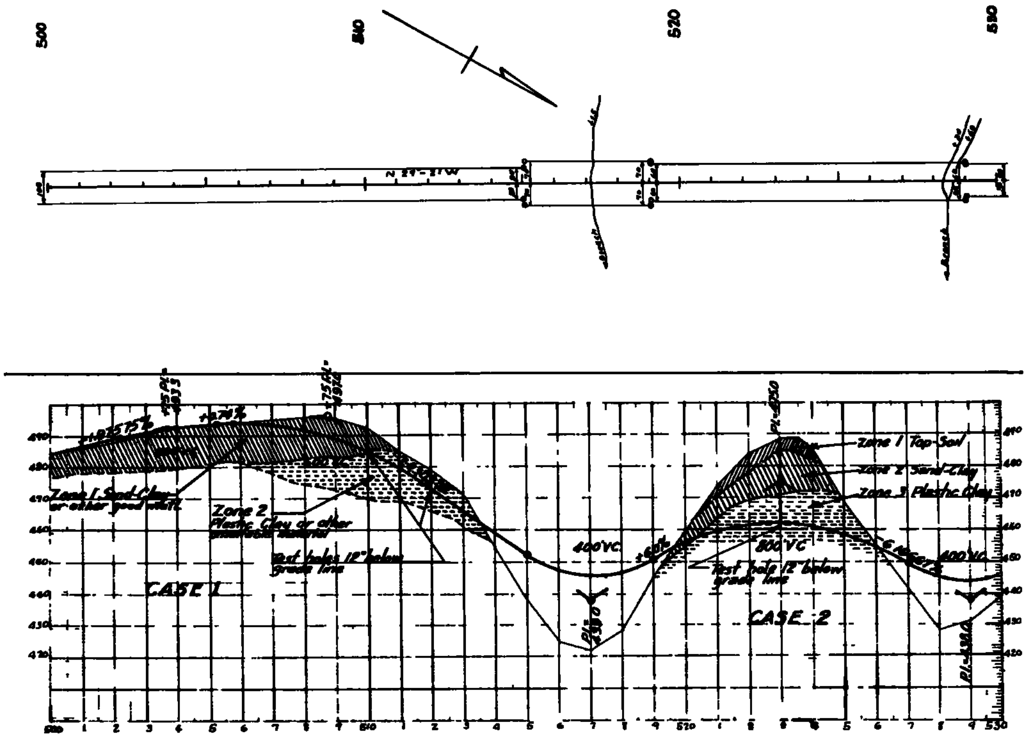


Figure C.

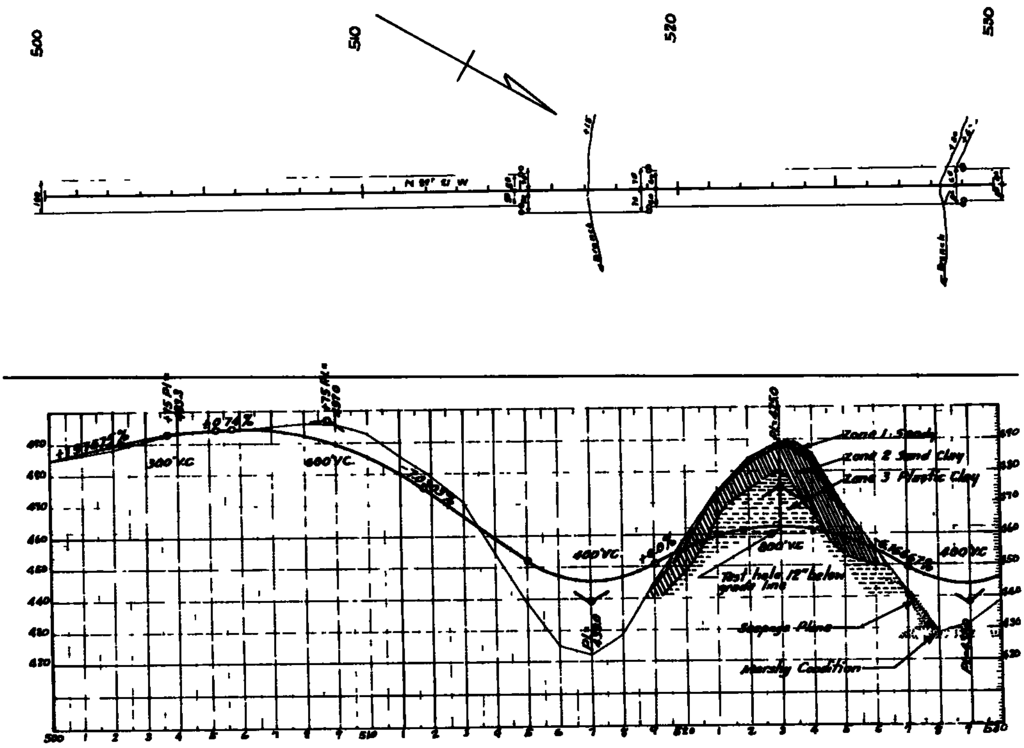


Figure D.

those materials which are common to the limits of the proposed project or adjacent thereto.

Sampling

1. Employ as a guide a set of plans for the project carrying alignment, profile and tentative grade line.
2. Take samples and make notes for each, being careful to record data for all types of soil encountered for use in the design of pavement. Record all marshy locations, springs, etc., that will fall within or affect the construction limits of the project.
3. Take samples from 2 to 5 station intervals or closer in cut sections as the case demands, to give complete information as to soil type or types encountered, being careful to take samples from those locations that will yield the information representative of the soil in place and submit the same to the testing laboratory.
4. a. Take a composite sample from ground profile line or surface to a depth equal to the difference between the elevation of profile and grade at some location if the soil appears to be uniform or of small variation (see Figure B).
   b. When the soil is stratified take a representative sample of each different type encountered and record the zone thickness of each type.
   c. Always make borings or dig holes as the case demands, at least 12 in. below the proposed grade line elevation as shown on plans, if possible, and if a change in soil type occurs from that immediately above take a sample and record notes on those phases.
5. Where hard-pans or other zones of impervious soils are encountered, make borings or dig holes as often as is necessary to determine its slope, water table, etc., for use in design and on construction (see Figures B, C, and D).
6. Fill out two sample information cards accurately for each sample taken, one to go with sample into container and the other in the envelope which is to be tied as a tag label on the outer side of the container. They should be identical and give complete information as to location on plans, depth of soil the sample represents and carry serial number. For instance, if a sample hole is bored at Station 523+00 its serial number is 23. If three zones of soil are struck, the first zone 2 feet, the second 4 and the third 3 feet thick, then the sample cards should be numbered 23-A, 0-2 feet deep; 23-B, 2-6 feet deep and 23-C, 6-9 feet deep.
7. Make borings or test holes at sufficient intervals to determine soil changes, regardless of whether samples are taken or not. (A superfluous number of samples represents waste but frequent examination of soil by eye and feel and making record of the same reflects alert engineering.)
8. Record in note book the depth of soils which each sample represents and any other information relative to soil type or ground water that could be considered of value in the design or construction of the project.

Analysis of Soil Samples

1. The samples are sent to the testing laboratory where they are analyzed in terms of standard specifications and in accordance with the information supplied and data requested on cards accompanying the sample.

Analytical reports are made for each sample in consecutive order as to the locations on the plans, etc.

Reports

Reports of laboratory findings or soil analysis and CBR values are made out in the testing laboratory office and copies of the same forwarded to parties concerned.

References

1. "Principles of Highway Construction as Applies to Airports, Flight Strips, and Other Landing Areas for Aircraft," Public Roads Administration, Federal Works Agency, June 1943, Section IV, "Flexible Bases and Surfaces."
2. "Engineering Manual" - Corps of Engineers, U.S. Army, June 1942, Chapter XX.

Adopted from California Method of Design Based on California Bearing Ratio.
3. "The Earth Mover and Boad Builder" by Arthur R. Smith, June 1938, to January 1939.
4. "Procedures for Testing Soils," by American Society for Testing Materials, September 1944.
5. "Suggested Information for Determining Base Course and Roadbed Topping," based on 1940 research by J. F. Tribble of Alabama Highway Department.
6. "Engineering Properties of Soil," by C. A. Hogentogler.

Appendix B

Material Surveys

Specific surveys are made for sources of local materials for use in construction and maintenance.

Purpose of Material Surveys

1. To predetermine by investigations, supported by analyses of samples, the types of local materials available, their working values, feasibility of use and economy of employment, for use in the design and construction of a given project.
2. To determine from (1) above the working value or suitability for use of the material from each source prospected.
3. To determine from (1) and (2) above the feasibility of use for materials from each source investigated.
4. To determine from (1), (2), and (3) above and from haul roads, etc., the economical factors involved by the use of materials from each suitable source.

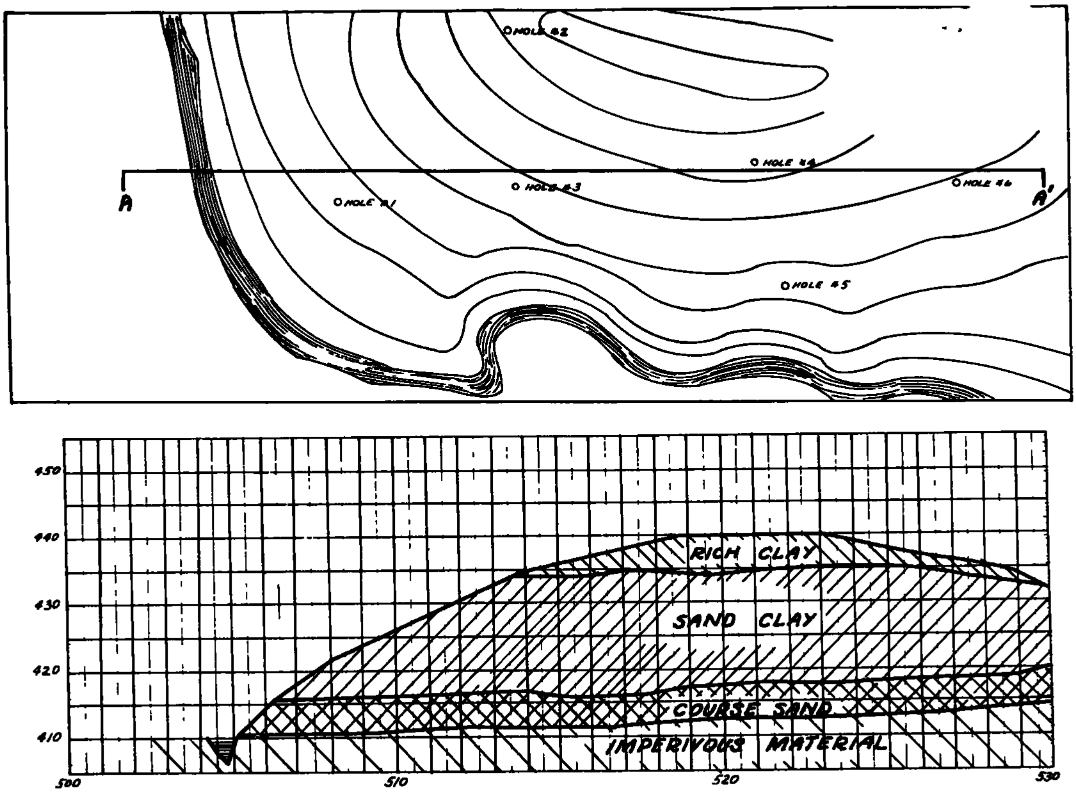


Figure A. Preliminary sampling of prospective pit (pilot samples) sand-clay.

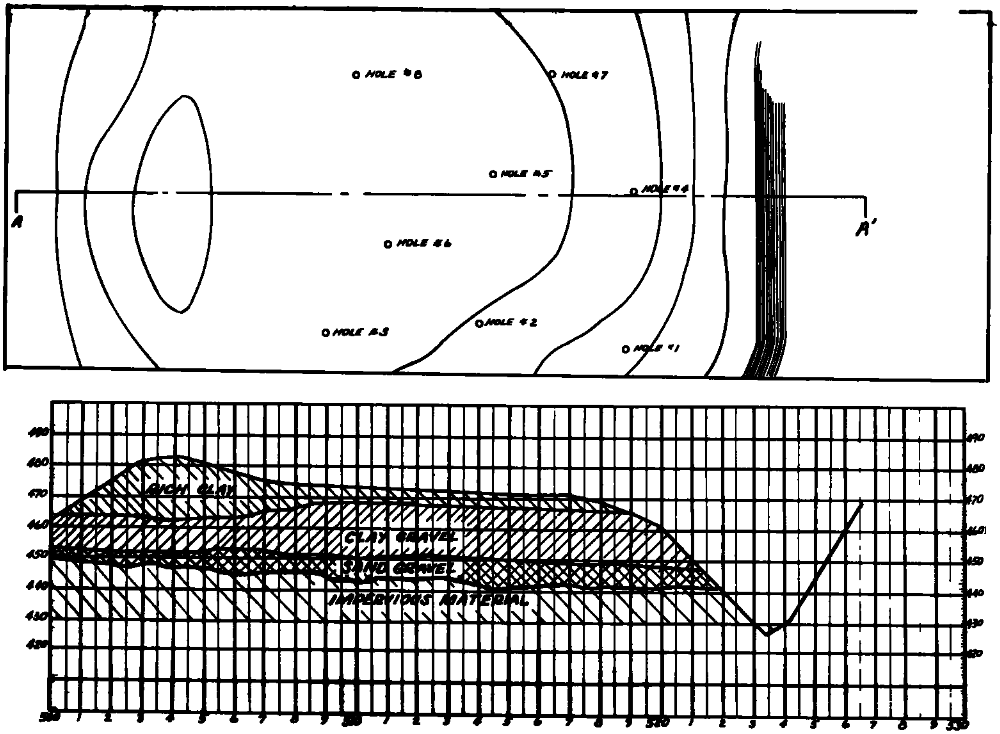


Figure B. Preliminary sampling of prospective pit - clay gravel.

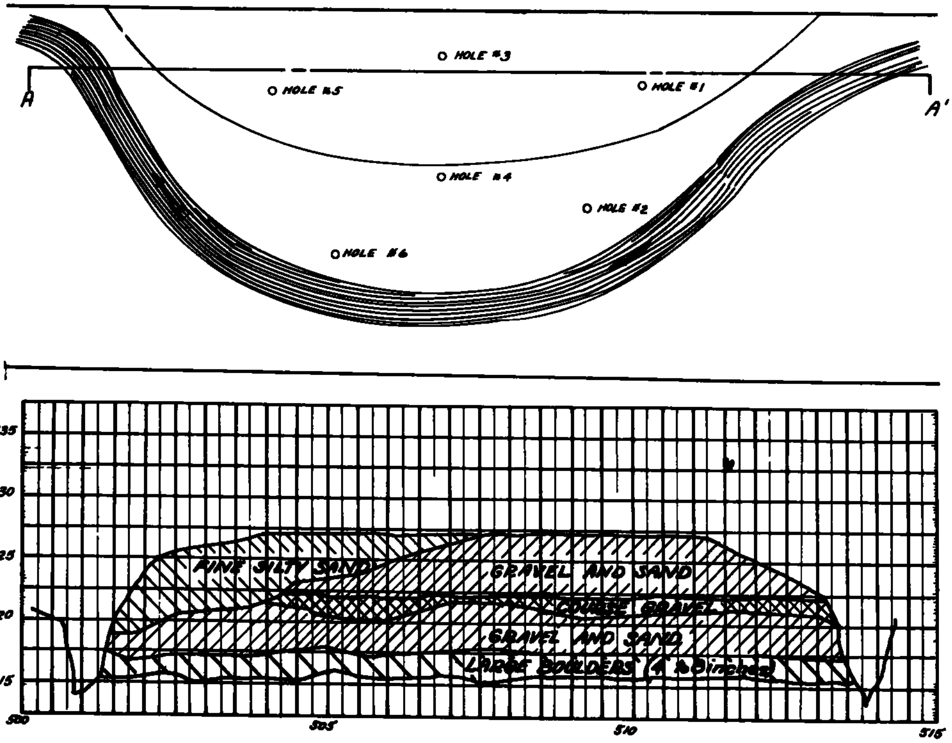


Figure C. Preliminary sampling of prospective pit (pilot samples) float gravel.

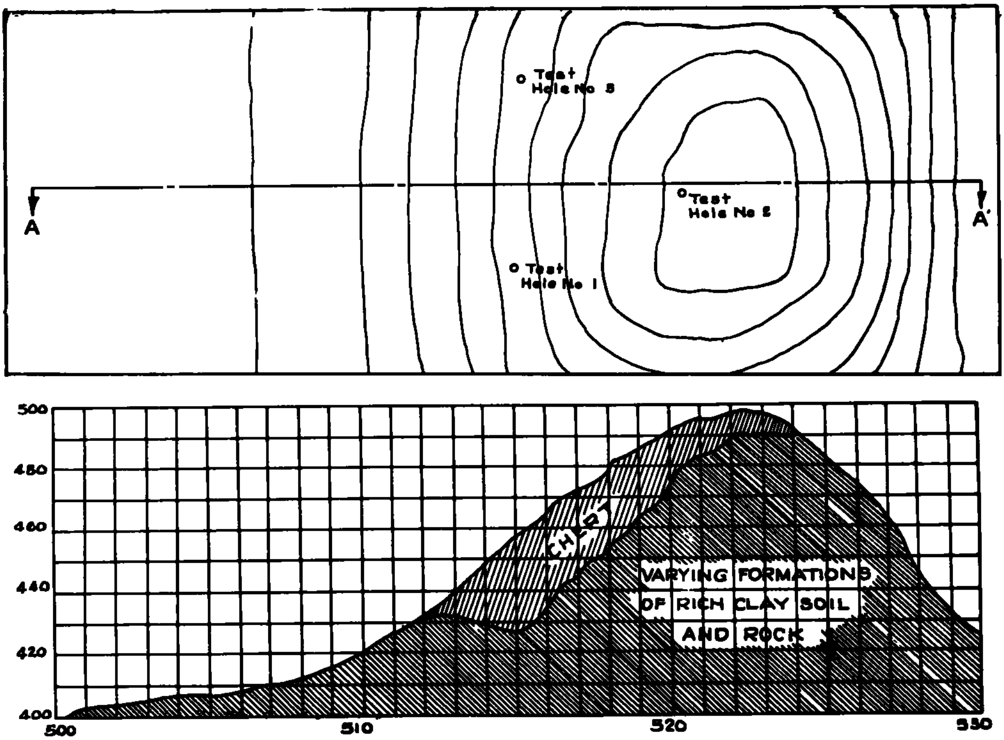


Figure D. Preliminary sampling of prospective source (pilot samples) chert.

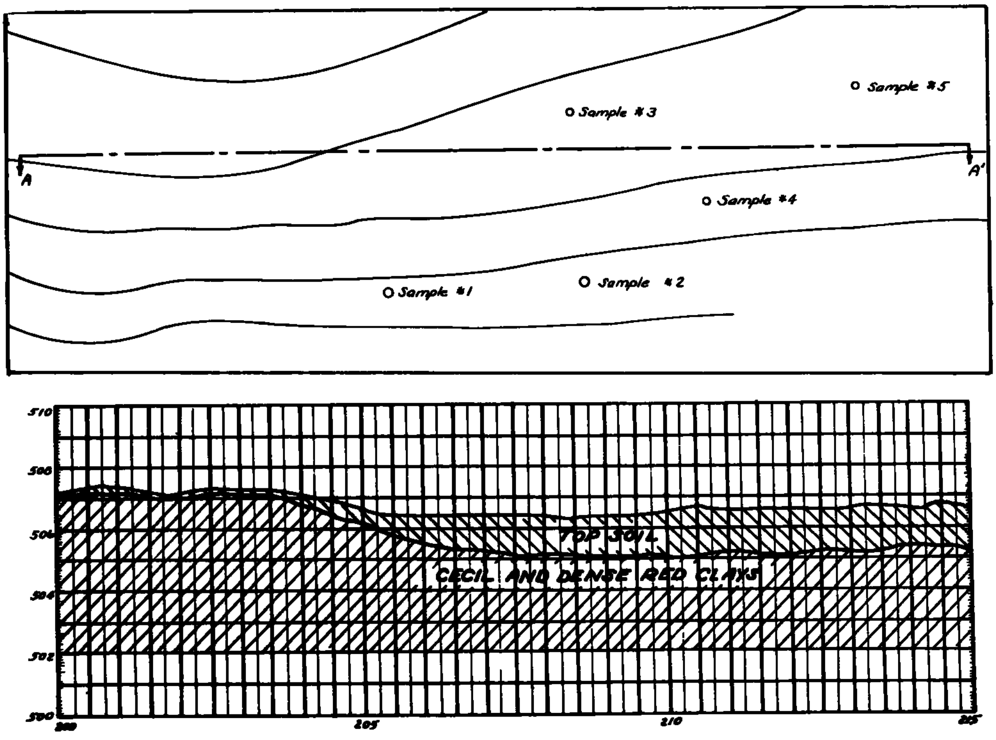
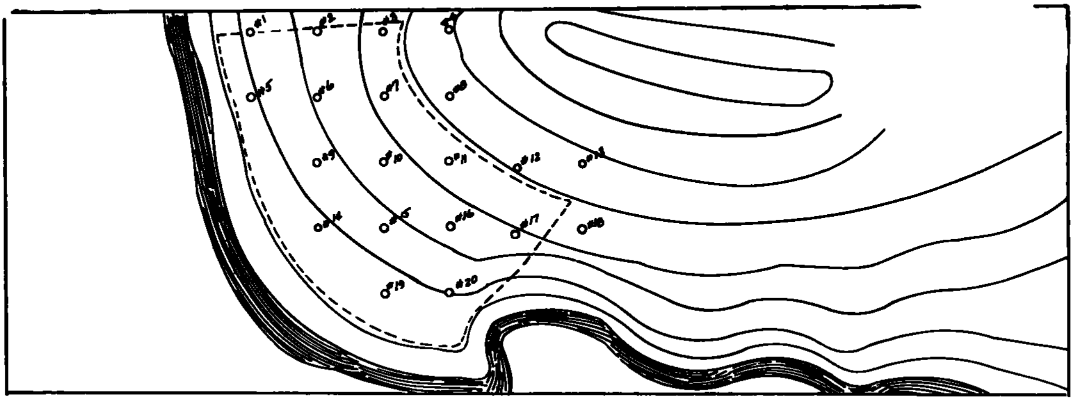


Figure E. Preliminary sampling of prospective source (pilot samples) top soil.

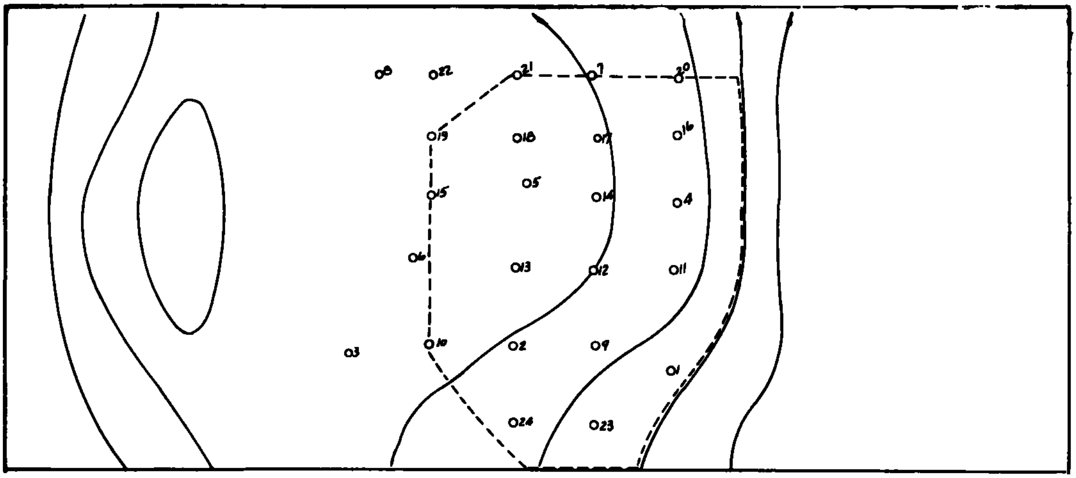
5. To select from (1), (2), (3), and (4) above the materials to be used in the design and construction of the project affected.

6. To analyze from (1), (2), (3), and (4) above the economical factors involving the use of local materials and determine their value as compared with commercial products when such products become competitive.



Note Test holes bored on approximately 100' centers to determine quantity and uniformity of available material. Material set-up for use shown in dotted outline, possibility of obtaining additional quantity of material at this particular site, but sampling only performed to amply cover quantity required for job

Figure F. Sampling of pit for quantity.



Note 1 - TEST HOLES bored on approx 100' centers to determine quantity and uniformity of available material Material set-up for use would be that shown in dotted outline Other not economical for use because of excessive stripping to be done for small yield of acceptable material

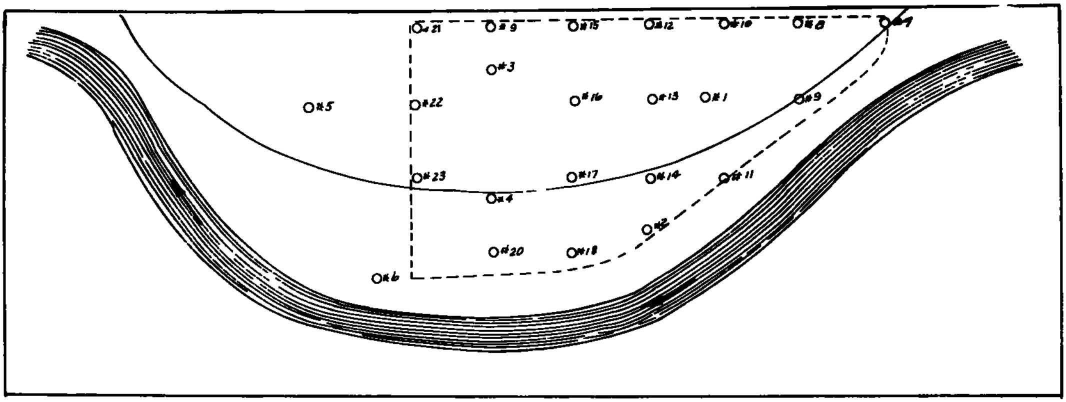
Note 2 - FREQUENTLY SAMPLING will show pockets or stratas of rich clay in a clay gravel deposit If these are not too large and there is sand or sand-gravel underlying the clay-gravel a satisfactory base course material meeting required specifications may be obtained by careful handling and mixing in the pit Care must be taken to go deep enough into the sand or sand gravel to compensate for the rich clay Mixing Must be Thorough

Figure G. Sampling of pit for quantity.

7. To determine availability of all sources prospected and any special conditions affecting their use, such as special requests of owners, etc.

Scope

1. Material surveys include complete investigations of known sources and extensive studies for the development of new sources of the materials that can be successfully employed in the construction and maintenance of highway projects. They will be made

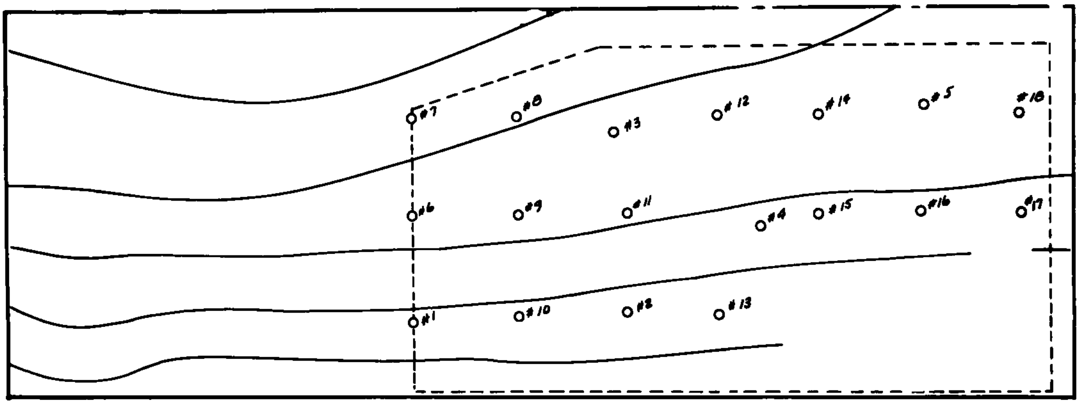


Note-1 Test holes bored (if possible) on approximately 100' centers to determine quantity and uniformity of available material In general soil augers cannot be used, making requisite, the digging of test holes with pick and shovel and in many cases the use of water pumps are essential Such deposits are often irregular in depth and require extra care in investigation for quantity Material set-up for use in this case would be that shown in dotted outline

Note-2 Water is usually encountered in this type of material and must be taken into consideration on plans for digging and loading

Note 3 Often this material is deficient in fine sand and will require the addition of such material to give the desired gradation

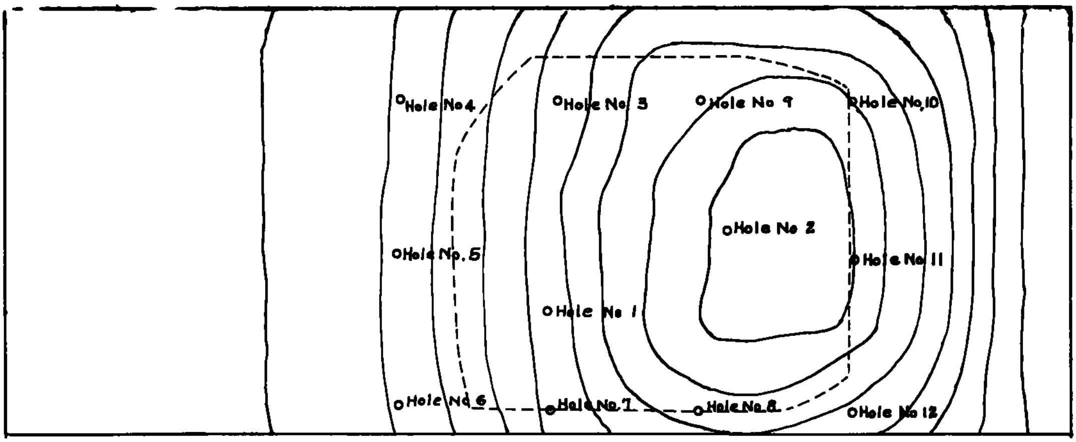
Figure H. Sampling of pit for quantity.



Note 1 .Test holes bored on appx 100' centers to determine quantity and uniformity of available material Material set-up for use would be that shown in dotted outline

Note 2 Topsoil commonly occurs 6 to 18 inches deep and usually requires windrowing or similar handling for economical loading.

Figure I. Sampling of source for quantity.



Note:1-Test holes Dug((Pick and shovel necessary)on approx. 100' centers to determine quantity and uniformity of available material. Material set up for use would be that shown in dotted outline

Note:2-Frequently pockets of rich clay and other undesirable materials that are not found on preliminary sampling will show up in chert deposits and must be avoided when loading out.

Note:3-Natural chert is generally deficient in well graded fine aggregate and usually contains an excessive amount of minus 200 Mesh material. This deficiency is usually corrected by the mixing in of coarse sand, slag, or limestone screenings.

Figure J. Sampling of pit for quantity.

in relation to soil surveys when feasible and advisable, and will supplement the same.

2. They are conducted in accordance with project demands and extend sufficiently to include soils for subbase and base courses, to paving and bridge aggregates, stabilizing materials either local or common to the vicinity or imported in accordance with feasibility of use and economy factors affecting the over-all cost of a project.

3. They are conducted by such intensive and extensive investigation that all usable materials for a given project are given consideration for use.

4. They give data on both materials suitable for use in their natural state, and those that can be processed satisfactorily for use either by manufacture or stabilization methods.

Method of Conducting Materials Surveys

General.

1. Employ as a guide for location, set of plans for the project carrying alignment, topography, profile, proposed grade line and a large county map.

2. Geological map of the affected area and a soil map of the county.

3. Obtain from soil survey data, geological formation, personal observation, or conference with other persons, the type of materials that can be reasonably expected to be found in the general areas to be investigated.

4. Compare notes with Bureau of Plans and Surveys, study soil data and arrive at a general conclusion as to the types of construction that will be employed and the materials requirements for the construction or maintenance of the project studied.

5. Armed with above suggested information and prospecting equipment, proceed with investigations for materials requisite to the design and construction of proposed work, or to the repairs on maintenance.

Prospecting.

1. Prospect all likely spots on the right-of-way, adjacent to the right-of-way, and in the general vicinity (within a reasonable haul distance) of the proposed project, for those

materials as mentioned in paragraphs above.

2. Being guided by practical knowledge and common reason, take pilot samples from each source of material found, considered of sufficient import to be investigated and submit the same to the testing laboratory for substantiating data. (See Figures A thru E for examples of pilot sampling of typical deposits.)

3. When the analyses of pilot samples are reported, complete the sampling of each acceptable source for quantity and quality (see Figures F thru J).

4. Much walking and hard work are necessary for making an intensive materials survey. A prospector must be alert and efficient and he must keep on looking for materials with a view of finding a source just 100 years beyond where he desires to turn around. Instances are on record where good material was found just over a hill from where another prospector got tired and stopped. Riding out all roads, trails and examining gulleys or natural breaks in terrain often renders many good sites. One should not exchange hard work and walking for the comfort of riding, if he expects to be a successful material survey man.

References

1. "Engineering Properties of Soil," by C. A. Hogentogler.
2. "Geological Survey of Alabama," by Eugene A. Smith.
3. "Soil Surveys of the Various Counties of Alabama," by the United States Department of Agriculture.
4. "Procedure for Testing Soils," by American Society for Testing Materials, September 1944.
5. "Principles of Highway Construction as Applied to Airports, Flight Strips, and Other Landing Areas for Aircraft," Public Roads Administration, Federal Works Agency, June 1943, Section IV, "Flexible Bases and Surfaces."
6. "Standard Specifications for Road and Bridge Construction," State Highway Department of Alabama and Other State Highway Departments Proceedings.
7. "Soil Mechanics and Soil Stabilization," Highway Research Board, Vol. 18, Part II, 1938.

Appendix C

Plant Mixes

Cold

Cold mixes yield many desirable features as to delivery, workability and placement, but present an ever constant problem of control for the specific purpose.

They are most generally employed for thin applications, ½ to 1 in. thick, and rolled lightly to facilitate losage of volatiles by evaporation, with the expectation of traffic completing density.

The gradation can not present too dense a mix, nor can a very high stability be expected immediately after placement.

We employ some 150 to 200 thousand tons annually for retreads or light surface courses over surface treatment.

Hot

Hot mixes are employed when economy, and situation permits its usage, and their manufacture and placement conform as closely with conventional standards as conditions demand.

Our available aggregate types are many, with specific gravities varying from 2. 15 to 2. 80, and weight from 80 to 120 pcf, respectively.

Little or no trouble is experienced when only one type of aggregate is employed. The gradation curve is normally regular and smooth, unless particle shape produces an excess of voids which have to be filled to produce desired density and stability (sand-gravel).

When two or more aggregates are employed, having different gravities, etc., a good surface area or volume curve for each sieve size may appear ragged, due to this item being secured by weight from sieve analysis. In such cases, the only way to secure a true gradation curve would be to determine specific gravity for each fraction between control sieve sizes and make corrections. This item is not too bothersome since we employ density and stability as a component of design.

To design a cure-all pavement is next to impossible. A retread over old concrete with a widened shoulder of flexible base is a striking example. The portion of surfacing over the concrete with non-yielding base will wear differently from that on the flexible widened strip. Too, a multiple laned highway may require different densities, stabilities and bitumen content for each lane with regard to traffic usage. Sometimes the bitumen content varies as much as 1½ percent between an outer and inner lane, to render equal service.

To serve the purpose intended, an asphalt surfacing must be smooth, not only for pleasant riding but to remove impact factors. Impact is very hard on light surface applications.

Economy comes from selection of cheapest suitable aggregates, so graded as to require only a reasonable amount of bitumen. The know-how to design mixes comes from laboratory studies, while the production of satisfactory results requires plant and placement control respectively, in the field.

ALABAMA METHOD OF DETERMINING AND CONTROLLING BITUMEN CONTENT FOR ASPHALT PLANT MIXES

The method of determining the bitumen content for bituminous mixes as herein below described is known as the surface area method and is to be used only for controlling the bitumen content of mixtures due to changes in gradation. The oil ratio derived by the method herein described must be multiplied by a factor k. The factor k to be used for any mix is determined after considering the shape, specific gravity, uniformity of gradation, and absorbtive qualities of all the mineral aggregate; the kind and specific gravity of the asphalt; the void content of the mix; the type and thickness of the proposed pavement; the stability and shear strength of the mix; the rainfall in the vicinity of the project; the volume and type of traffic; the kind and quality of the base course; the drainage conditions; and texture required of the finished pavement.

The method requires the use of surface area factors and a chart (Figure A) for converting total surface area of the combined mineral aggregate to obtain an oil ratio figure. Figure A gives surface area factor for the seven sieve sizes which have been selected for determining the surface area of the combined mineral aggregate and also gives a curve showing the relation of surface area and oil ratio.

The oil ratio is defined as the weight of bitumen used divided by the weight of the aggregate, the result being multiplied by 100.

The oil ratio is obtained by making a sieve analysis of the mineral aggregate in the total mix; calculating the aggregate retained between each sized sieve; multiplying this by the surface area factor for each size sieve; and adding the surface area for each sieve to get the surface area of the mix. Table A shows how to obtain the surface area of the mix.

COLUMN A — gives the sieve sizes used.

COLUMN B — is tabulation of the regular sieve analysis. The ⅜ in. sieve is not considered for the purpose of this formula. All the material between the ½ in. and No. 4 sieves are grouped together using the same surface area factor of 3. 2.

COLUMN E — gives the surface area factors which are used regularly for each fraction of the material that passes one sieve and stays on the next. These figures are constant and are used for all cases.

COLUMN F — is the result of multiplying Column D by Column E. In Column F the decimals are neglected, the numbers being entered to the nearest whole number.

TABLE A

EXAMPLE OF CALCULATION OF SURFACE AREA

Sieve Sizes	% Passing Cumulative	Weight of Material Between Sieves: Sieve Sizes	%	Surface Area Factor	Surface Area
A	B	C	D	E	F
½"	100.0				
⅜"	81.1				
No. 4	54.3	½ - 4	45.7	3.2	146
No. 10	38.2	4 - 10	16.1	6.4	103
No. 40	14.4	10 - 40	23.8	19.8	471
No. 80	5.5	40 - 80	8.9	81.5	725
No. 200	2.1	80 - 200	3.4	182.0	619
		Minus 200	2.1	615.0	1,292
		Totals	100.0		3,356

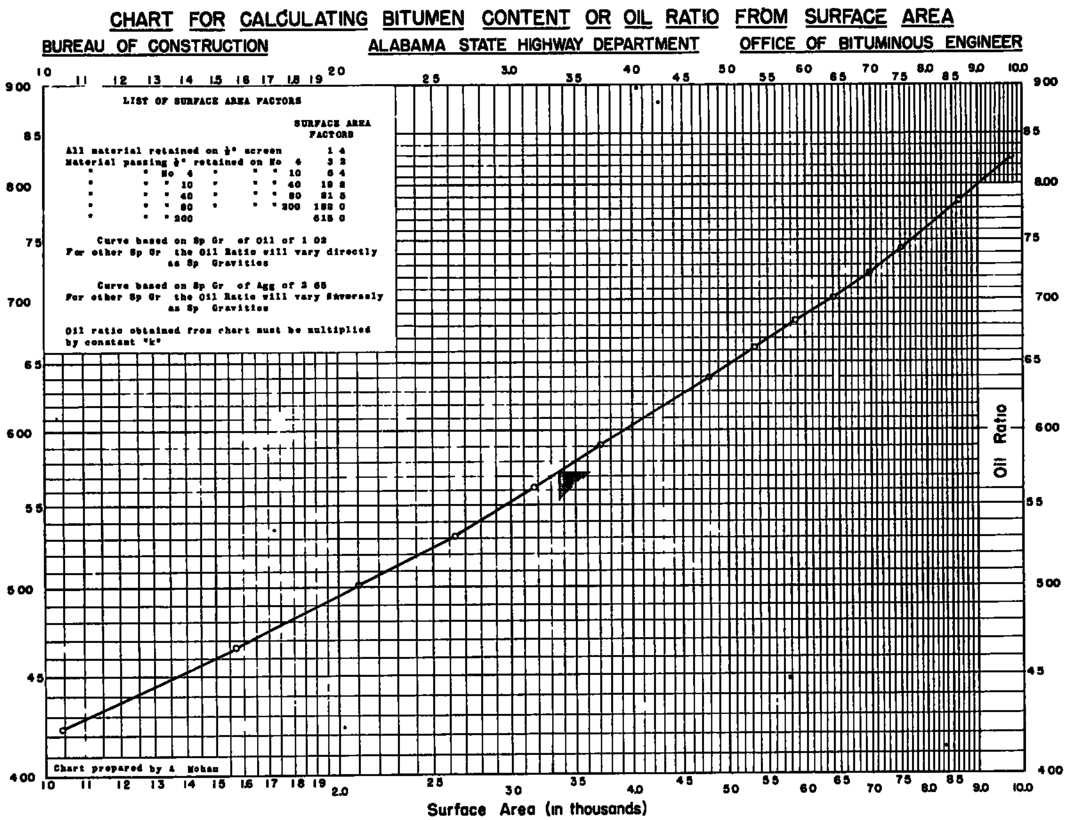


Figure A.

In Column D, the figure 45.7 is obtained by subtracting 54.3 from 100.0; 16.1 is the result of subtracting 38.2 from 54.3; and so on. 2.1 is all the material passing the No 200 mesh sieve.

The total surface area so obtained is projected on the chart as shown. In the example noted above the total surface area was 3,356, which expressed in thousandths to agree with the chart is 3.356, or, to the nearest second decimal, 3.36. The vertical line 3.3 is found on the chart and six tenths of the distance between lines 3.3 and 3.4 is estimated. Opposite where this estimated line cuts the heavy diagonal line (marked with a hatched angle on chart) the oil ratio is read at right hand side of chart. In the given example it is 5.73, the second decimal "3" being estimated between lines 5.7 and 5.8.

After the oil ratio is so obtained it must be multiplied by the factor "k" to obtain the bitumen content. The factor "k" for each project will be set by the state bituminous engineer only. The bitumen content is the percentage of bitumen of the total amount of mineral aggregate and may be expressed as the number of pounds of bitumen per 100 lb of mineral aggregate.

In the example above 0.9 was selected as the factor "k," hence the bitumen content to be used in the mix is 5.73 percent (oil ratio from Figure A) x 0.9 or 5.16 percent which is the corrected oil ratio. If the mix is to be made using 2,000 lb of mineral aggregate per batch then the amount of asphalt to be used will be 200 x 5.16 percent or 103.2 lb.

If the total batch is 2,000 lb then the amount of bitumen as per content noted above would be 98.1 lb obtained as follows:

$$x \text{ (amt bitumen per batch)} = \frac{\text{wt of batch} \times \dfrac{\text{corrected oil ratio}}{100}}{1 + \dfrac{\text{corrected oil ratio}}{100}} = \frac{200 \times 0.0516}{1.0516} = 98.1 \text{ lb}$$

and the amount of aggregate to be used would be 1901.9 lb obtained as follows:

$$y \text{ (amt of aggregate per batch)} = \frac{\text{wt of batch}}{1 + \dfrac{\text{corrected oil ratio}}{100}} = \frac{2,000}{1.0516} = 1901.9 \text{ lb}$$

Discussion

RAYMOND C. HERNER, Chief, Airport Division, Civil Aeronautics Administration, Indianapolis — As the ultimate test of all design theories must be found through their application in the field, we are indebted to all those who have taken the time and trouble to acquaint us with their current practices.

Land's paper was of special interest because his organization uses two distinctly different approaches in evaluating subgrades for determination of paving thicknesses. Table A of this discussion was prepared for the purpose of comparing the results of these two methods. It lists the group index and CBR values for each test location, the design thicknesses determined by each method, and the thickness of section actually built. Design assumptions are given in a note at the end of the table.

Although there is a general correlation discernible between the values of CBR and the group index, the design thicknesses obtained from their use vary widely. The CBR thicknesses are the lower in all instances. In the first two projects the section actually constructed is very close to that obtained from the group index, with the CBR apparently given little consideration. In the third project the as-built section is more nearly an average between the theoretical thicknesses obtained from the two methods.

The as-built section in the first project is of uniform thickness throughout the length of the project despite rather extreme variations in indicated subgrade strength. In the third project, however, thicknesses were varied in accordance with subgrade test values.

The above observations may indicate that the Alabama design method is still in a state of flux. They definitely suggest a strong blending of the time-honored "experience and engineering judgment" along with the two formal design methods. This is not necessarily a reason for condemnation, as there still are many important factors which have not been thoroughly evaluated in any design approach. As someone has well said "A formula, carefully followed, always gives the same answer, but it may not be the correct one."

Unfortunately, the projects reported upon by Land are too new for any worthwhile service evaluation. It will be very helpful if he follows up with a sequel to this paper a few years from now.

TABLE A

DESIGN SUMMARIES

Project U-352 (1) Montgomery Co						
Sta No.	42	120	184	258	287	Avg
Group Index	3	2	15	20	12	10
CBR	7.7	14.9	7.8	3.2	5.0	7.7
Design Thickness [a]						
By G.I.	16	14	23	24	21	20
By CBR	12	8	12	20	15	13
As Built	20	20	20	20	20	20

Project F-286 (1) Madison Co						
Sta No.	27	57	92	152	200	Avg
Group Index	10	9	5	9	9	8
CBR	12.0	13.2	11.0	13.8	12.7	12.5
Design Thickness [a]						
By G.I.	20	20	17	20	20	19
By CBR	10	10	10	10	10	10
As Built	16	16	16	16	16	16

Project F-120 (3) Bibb Co						
Sta No.	230	290	375	575		Avg
Group Index	0	10	8	8		6.5
CBR	28.3	7.3	8.3	7.0		12.7
Design Thickness [a]						
By G.I.	12	20	19	19		18
By CBR	6	13	12	13		11
As Built	11	17	15	17		15

[a] Surfacing not considered or included. Group index design based on "Heavy Traffic" chart (Curve D); CBR design based on 9,000-lb wheel load.

J. L. LAND, Closure — This paper was prepared from current material employed in Alabama State Highway Department school classes, for the sole purpose of this panel discussion. The three examples were injected to develop just such points as Herner has analyzed.

It later developed that only a brief period could be allocated to this series and the paper resulted in a mere outline, since it would require several pages to explain the reason for actual set-ups employed.

An attempt was made to explain:

1. Usage of local materials, both as to feasibility and economy. This usage dictates the employment of the most suitable material available having the required structural value desired. Sometimes a material three times the strength required is employed in a particular layer of foundation course because it is readily and economically available.
2. All foundations are set up in the field, on the job, taking into consideration soil, grade line, drainage, local materials available, sub-drainage, type of base course, and traffic demands.
3. Typical sections for weak, medium and strong bases.

Each project is considered a completely new problem within itself and all the factors affecting its design and construction in the field for its entire length are employed.

Roadway pavement is designed for each five-station interval and averaged for short sections, taking into consideration the weakest portions of each section. Averaging of group indices from 0 to 10 on roadway evaluation seems unthinkable.

Flexible Pavement Design in Washington

ROGER V. LECLERC, Senior Materials Engineer
Washington State Highway Commission

This paper described the design procedure for determining the total depth of cover used over subgrade soils for flexible pavements. The design procedure is based on the Hveem stabilometer test. A brief history of the use of this test is given, and the various steps in the design procedure are outlined. Included are descriptions of the preparation and testing of soil samples, the analysis of test data to determine surfacing depth requirements, and the modification of these surfacing depths where cement treated bases are used. Special handling of swelling soils is also described.

Copies of surfacing design curves, photographs and sketches of test equipment, copies of completed test data sheets, and a table of typical surfacing requirements for various classes of subgrade soils serve as illustrations.

● PRIOR to January of 1951, flexible pavement design in the Washington Department of Highways was based on the California Bearing Ratio Test. This test was abandoned in favor of the Hveem stabilometer test, however, because with certain soils, notably clayey gravels and clean sands, it was difficult, if not impossible, to obtain reliable test results — results which would correlate with the observed performance of these materials in the roadway. In addition, the necessary time requirements of the CBR test limited our testing capacity to such an extent that it was impossible to handle the increasing number of samples being received from our expanded construction program.

The present design procedure for flexible pavements is essentially that originated by the California Division of Highways as outlined in the paper by Hveem and Carmany, "The Factors Underlying the Rational Design of Pavements" (1). The principal differences in procedure stem from certain modifications in test conditions and in factors used to evaluate the worth of base and pavement courses. These modifications were incorporated to give final design figures which are compatible with observed field conditions and roadway performance in the Washington highway system.

The method of design is based on the layer "theory" which holds that each component of the roadway section must have better load-supporting ability than those components under it, and that the surfacing or cover requirements of each must be satisfied. The grading, fracture and cleanliness requirements of certain of these surfacing components (crushed stone or crushed gravel surfacing, ballast, etc) are controlled by specification, and minimum cover requirements have been assigned on the basis of numerous stabilometer test data. The required surfacing depths of other select, local, cover materials are determined by stabilometer tests on representative samples submitted in the preliminary stage of the roadway design. Types and minimum thicknesses of bituminous pavements for use on the various classes of highways are set forth in the design standards, having also been determined from accumulated test and performance data on these specification materials.

The procedure by which the total depth of surfacing (including bituminous mat) is determined for any one soil involves, first, a determination of the index of its load supporting ability by means of the Hveem stabilometer. This index, the stabilometer "R" value, is then converted to a total surfacing depth requirement through use of a traffic factor, and this total depth is revised downward in recognition of the limited slab action or stiffness of a cement treated base course if such is used. Inasmuch as our current design standards specify the use of a cement treated base under flexible pavements on the three principal classes of highways, this downward revision of surfacing depths is a major consideration in the design procedure. The individual steps in the design procedure are described more fully in the following sections.

Soil Sampling and Testing

During the soil survey for any location, the district soils engineer and his crews determine the location and extent of each soil type to be encountered on construction.

This includes test drilling all cuts to and beyond proposed grade elevation. Representative samples of each different soil are taken and submitted to the laboratory.

Routine tests performed on these soil samples are: mechanical analysis, Atterburg

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS
Materials Laboratory

To: Planning Division — Date 1-4-55

From: Materials Laboratory — By WAS

Subject: Traffic Data

Please furnish listed traffic data for the following location:

P S.H. No. 12 Section Cathlamet to Skamokawa

Job No. L-865 Sta. Limits 68+00 to 194+00

Control Section 3502

Yr.	ADT
1955	1140
1965	1600

Truck Classification (% of ADT):

% 2 Axle (Exclusive of Pickups & Panels)	11.6
3 Axle	3.5
4 Axle	0.8
5 Axle	1.5
6 Axle	0.3

Remarks: * None

* Please note any unusual characteristics of traffic, particularly if one lane will be subjected to heavier loads and/or heavier volumes. In this event, please furnish ADT and classification for each lane or direction. Surfacing design is based on the heaviest traffic conditions to be expected in any one lane.

Figure 1.

WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS
Materials Laboratory

EWL* COMPUTATION

Date 2-2-55

By RVL/WAS

*Equivalent 5000 lb. Wheel Loads

P S.H. No. 12 Section Cathlamet to Skamokawa

Sta. Limits: 68+00 to 194+00 Job No. L-865

C.S. 3502

Traffic Data

Yr.	ADT
1955	1140
1965	1600

Truck Classification (Exclusive of Pickups & Panels)

By Axles:

2 Axle:	11.6 %	x	3,000 =	348
3 " :	3.5	x	7,000 =	245
4 " :	08	x	14,000 =	112
5 " :	1.5	x	21,000 =	315
6 " :	0.3	x	16,000 =	48

Item(1): 10 Yr. EWL/ADT = 1068

ADT Adjustment for Lane Width & Increase of Traffic

No. of Lanes: 2

ADT for current yr. : 1140
ADT for current yr. + 10: 1600
Average ADT : 2740 ÷ 2 = 1370

Average one-way ADT: 685

Item(2): Average one-way ADT on heaviest lane: 685

EWL = Item(1) x Item(2) = 1068 x 685 = 731,580

Design Curve C

Figure 2.

limits, moisture-density, and stabilometer "R" value. The details of the latter test are covered in Appendix A. Our procedure for this test differs but slightly from that used by the California Division of Highways (1). The soil specimen is compacted in a triaxial institute kneading compactor, subjected to a vertical pressure until saturation is indicated by the exudation of water, examined for swell pressure, and tested in the Hveem stabilometer.

Evaluation of Traffic

One of the first steps in the design procedure for any particular project is the determination of the maximum traffic to which any lane of the proposed roadway will be subjected during the 10 years following construction. Coincident with the receipt of soil samples from any location the Planning Division is requested to supply complete traffic information to the Materials Laboratory. The standard form for this request is shown

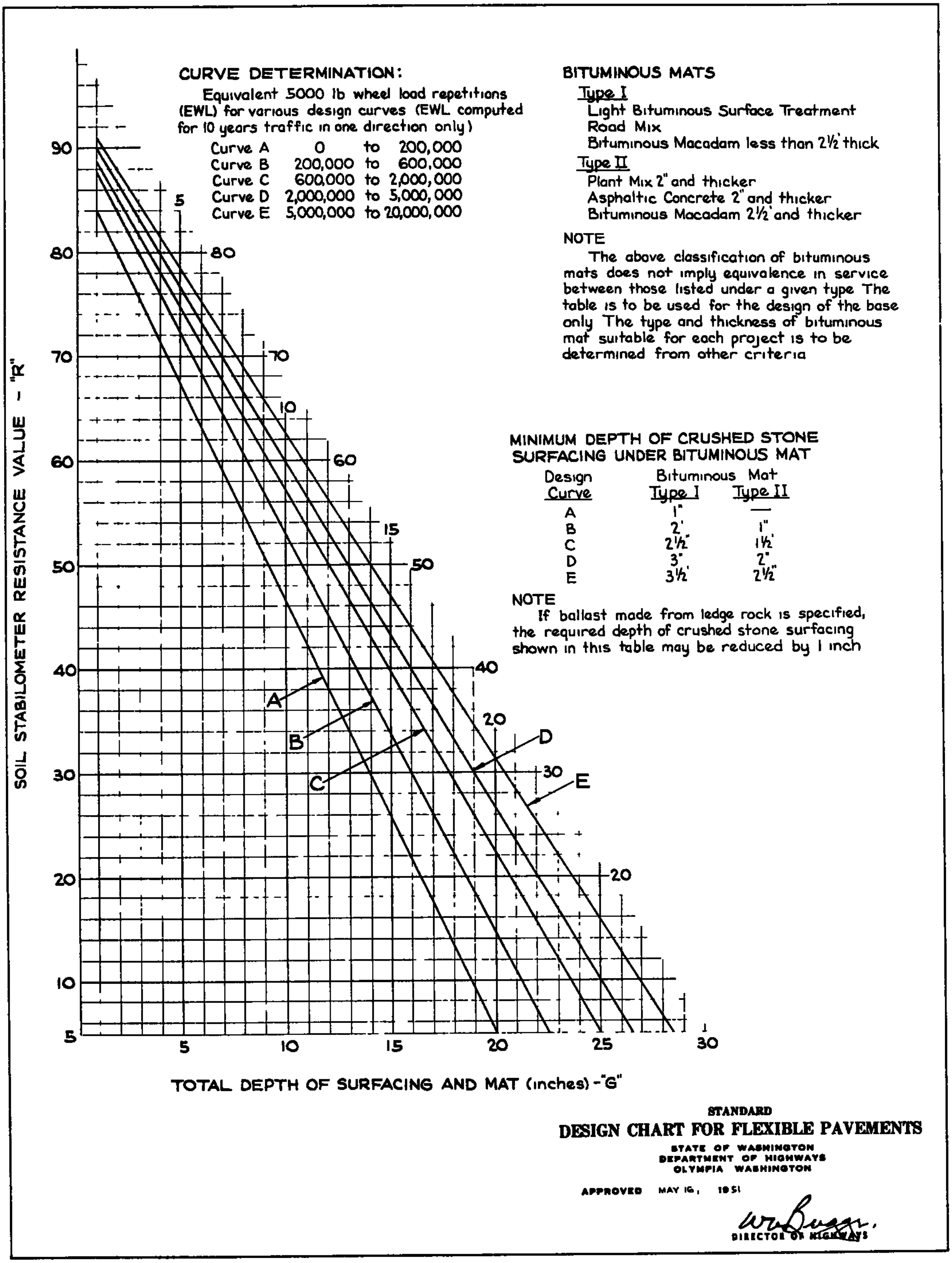


Figure 3.

in Figure 1 with entries of typical values. The data supplied on this form give a classification of commercial vehicles, excluding pickups and panels, according to the number of axles. This classification is given in terms of percent of average daily traffic. Also given are data from which may be calculated the average daily traffic in the most heavily traveled lane for the design period of 10 years.[1] This information is converted to equivalent 5,000-lb wheel loads (EWL) by means of the following conversion factors:

No. of axles per truck	Conversion Factor (No. of equivalent 5,000-lb wheel load repetitions per 10 yr per daily pass)
2	3,000
3	7,000
4	14,000
5	21,000
6	16,000

A sample calculation is shown on the work sheet, Figure 2, using traffic data given in Figure 1. The EWL thus calculated determines which of the five surfacing design curves is to be used, according to the limits shown under "Curve Determination" in the upper left of the Standard Design Chart for Flexible Pavements, Figure 3.

The Standard Design Chart consists of five curves relating stabilometer "R" values to the total depth of surfacing and bituminous mat required for different traffic intensities. The curves were established from approximate design figures obtained by California highway design formulas and adjusted to conform to established performance and service data in the Washington highway system.

Determination of the Surfacing Requirements for Individual Soil Sample

The data from individual tests on the four specimens of any one sample include the "R" value, the exudation pressure, and the swell pressure. The total amount of surfacing required by "R" value considerations is obtained from the standard surfacing design chart for flexible pavements, Figure 3. This is called the "gravel equivalent." The depth of surfacing necessary to restrain the material from swelling is determined by weight considerations alone. An average unit weight of 144 pcf is assigned to our surfacing materials, and a depth sufficient to produce pressure equal to the swell pressure is computed. This is termed the "swell equivalent."

The gravel equivalent is plotted against the swell equivalent. The intersection of the resultant curve with a diagonal line drawn through points of equivalent depths is taken as the equilibrium point where both surfacing requirements are equal. This surfacing depth is called the design depth determined by swell. In addition, the gravel equivalent is plotted against the exudation pressure and the value at 400 psi is designated as the design surfacing depth determined by "R" value. The greater of these two figures is used as the total surfacing depth or cover requirement of the material being tested.

A copy of one of our standard test sheets, complete with stabilometer test data and surfacing depth determination curves is shown in Figure 4.

Surfacing Depth Recommendations for Projects

Subsequent to the soil survey and submission of samples to the laboratory, the district soils engineer prepares a soils profile for the location, incorporating all the field data relative to soil types and their extent. The soils profile, together with a field soils report is transmitted to the laboratory. The standard outline for the field soils report is given in Appendix B.

The surfacing depth recommendations for the project are made after a study of the laboratory test data, the recommendations in the field report, and the in-situ position of soils shown on the profile. If it is indicated that selective placement of the better soils is feasible, recommendations are made for such procedures with surfacing depths

[1] There is oftentimes a major difference in the directional traffic density on highways which are used to carry timber, mining or agricultural products to market.

being governed by the requirements of the individual soils. If this is not the case, recommended surfacing depths for the project are based on the requirements of the weakest soil which may form the subgrade.

H F No 26 80—(Rev)

WASHINGTON DEPARTMENT OF HIGHWAYS

MATERIALS LABORATORY

SOIL SAMPLE TESTS

Job No L-865
PSH No 12
Section Cathlamet to Skamokawa
Sample No 10

Field Description Yellow-Brown Clay-Silt

Lab No 5-2191
Bin No 306
Date Rec'd 1-4-55

GRADING ANALYSIS

RETAINED Sieve	Wt	%	PASSING Sieve	%	%	AS USED Size	%	Wt
1½"			1½"					
1"			1"			¾"—⅜"		
¾"			¾"					
⅜"			⅜"			⅜"—#4		
#4			#4	100				
Pass #4	41.6	100	10			Pass #4	100	1000 gms
Total			40					
Est Fract			200					
Date (Coarse) 1-21						Specific Gravity		
Date (Fine)						Textural Class		

Operator CONSTANTS

	Liquid Limit	Plastic Limit	
Can No			
Wet Wt			
Dry Wt			
Wt H_2O			
% H_2O			
No Blows			
L L			
P L			
P I			

HRB Class Group Index

Operator NH STABILOMETER TEST Date 2-8-55

90 cc Temp H_2O	A	B	C	D
Cc H_2O added	60	45	52	38
% H_2O added	6.6	4.9	5.7	4.2
Initial % H_2O	21.1	21.1	21.1	21.1
Molding % H_2O	27.7	26.0	26.8	25.3
Molding Density	94.4	96.8	96.1	97.8
Compactor pressure	100	100	100	100
No blows	40	40	40	40
Wt in mold	3219	3148	3169	3161
Wt in mold (soaked)	3224	3152	3174	3169
Wt of mold →	2204	2154	2152	2166
Net wt of soil	1015	994	1017	995
Height	2.55	2.47	2.53	2.46
Exudation pressure	280	490	390	560
Swell pressure	1	31	16	55
Drainage	—	—	—	—
Stabil "Ph"— 500#	17	11	13	10
" —1000#	42	24	28	21
" —2000#	108	64	76	63
Displacement "D"	2.54	2.75	2.75	2.84
"R" value	32	58	50	64
Gravel equivalent	17½	9½	12	8
Swell equivalent	½	15	7½	26½

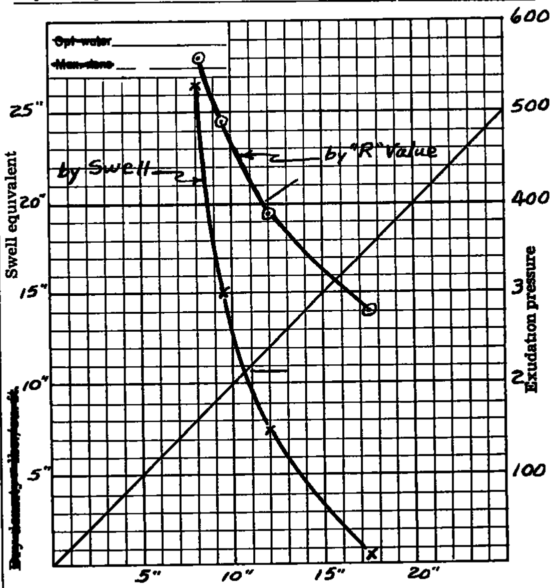
REQ'D SURF DEPTH By "R" 11½" By Swell 10½"

TRAFFIC	DESIGN "R"	52	
C	Swell Pressure	—	psi
	Surfacing Depth	11½	in

Remarks

MOISTURE-DENSITY (AASHO T-99-38)

Hyg Water Cont		%	Date		Operator	
Can No						
Est % H_2O						
Spec wt						
Wet wt	1108					
Dry wt	915					
Wt H_2O	193					
% H_2O	21.1					
Dry density						



Lab No 5-2191 Bin No. 306

WHITE COPY—Grading & Constants
PINK COPY—Moisture-Density
YELLOW COPY—Stabilometer

Figure 4.

Modification of Surfacing Depth for Cement Treated Base

Cement treated base consists of manufactured or processed mineral aggregates and portland cement uniformly mixed, moistened, and compacted to a specified thickness, density, and roadway section. The aggregate is usually a 1-inch minus product; the cement content varies generally from 3 to 6 percent by weight of aggregate; and the design compressive strength is 650 psi at 7 days.

The standard cement treated base section consists of a 6-inch compacted depth of treated material. Asphaltic concrete or plant mix with a minimum thickness of 3 inches is used as the pavement over this base.

In the event that cement treated base is to be used beneath the flexible pavement the total surfacing depth, determined as described previously, is revised downward according to the following formula:

$$S_M = \frac{S_T}{\sqrt[5]{C}}$$

Where S_M = Modified surfacing depth
S_T = Total surfacing depth
C = Correction factor

The correction factor is based primarily on the cohesiometer values and relationships used by the California Division of Highways, except that somewhat lower equivalent values are used. The modified surfacing depth for the standard cement treated base section and 3 inches of asphaltic concrete is obtained from the curve shown in Figure 5. Correction factors used in constructing this curve are equivalent to cohesiometer values of 600 for cement treated base and 250 for asphaltic concrete if the California equations and relationships are used.

Special Design Considerations Involving Swelling Soils

When tests on preliminary samples indicate that an appreciable quantity of subgrade soils on any project are subject to excessive swell pressure, and when it is apparent that paving construction will not occur during the same season as the grading, departure from the usual design procedure is usually recommended. This is done to prevent loss of subgrade compaction during the interim wet season, or to provide means for regaining it if it is lost through swelling action.

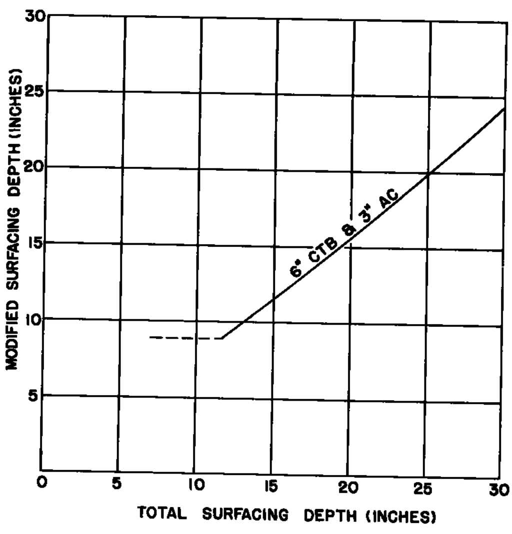


Figure 5. Surfacing depth reduction for cement treated base.

If a portion of the surfacing or cover courses is to be placed during the grading operation, the depth of this course must be sufficient to restrain swell pressure, and such depths are based on the soil tests. This sometimes results in an ultimate overall surfacing depth greater than necessary. However, this over-design may be warranted by other considerations such as the necessity of maintaining a large volume of construction or other traffic, the possibility of time limitations on the working of materials sources, or the desirability of stage construction involving separate contracts for paving and grading.

When only the subgrade and none of the surfacing is to be completed during the first season a density survey is made at the beginning of the next construction season. If loss of density is evident, the top lifts of completed subgrades are loosened and recompacted before placing the surfacing courses.

TABLE 1

TYPICAL TEST VALUES AND SURFACING DEPTHS - WASHINGTON SOILS

Material	HRB Class	Stabilometer "R" Values	Total Surfacing Depths Light Traffic in.	Med Traffic in.	Heavy Traffic in.
Silty and sandy gravels, gravelly silts and sands	A-1	40 - 84 (131)	11½ - 1	15 - 2½	17½ - 3½
Sands	A-3	66 - 72 (11)	5½ - 4	7½ - 5½	9 - 7
Silty sands, sands and gravelly silty sands	A-2-4	30 - 79 (60)	14 - 2½	18 - 3½	20½ - 5
Gravelly clay-sands, silty sands, and sandy silts	A-2-5	45 - 79 (12)	10½ - 2½	13½ - 3½	15½ - 5
Sandy clayey gravel	A-2-6	33 - 81 (18)	13½ - 1½	17 - 3	19½ - 4
Gravelly clays and gravelly sandy clays	A-2-7	18 - 79 (19)	17 - 2½	21½ - 3½	24½ - 5
Sands, silty sands, sandy silts and clay-sands	A-4	8 - 76 (89)	19½ - 3	24 - 4½	27½ - 6
Sandy silts and clay-silts	A-5	33 - 62 (29)	13½ - 6½	17 - 8½	19½ - 10
Silty clay and clay-silts	A-6	5 - 47 (17)	20 - 10	25 - 13	28½ - 15
Clays, silty and sandy clays	A-7	6 - 50 (45)	19½ - 9½	24½ - 12	28 - 14

Parenthetical numbers indicate the number of samples composing each group.

Typical Total Surfacing Depth Values

Table 1 gives a range of total surfacing depth values for various Washington soils tested according to the procedure described heretofore. The figures are taken from laboratory test data and represent materials tested within the last year. The range in "R" values for some soils may seem somewhat anomalous, but this is probably due to the limited number of samples represented. The number of samples involved in each range of values is shown in parenthesis.

Evaluation of Design Procedure

The Hveem stabilometer test has been the basis for flexible pavement design in the Washington Department of Highways for nearly 5 years. It has proved to be a test that can be conducted on a production basis and performed satisfactorily and efficiently by laboratory technicians.

While there is a background of only slightly over 4 years' experience by which to gauge its merits, the performance data are encouraging. To date no roadway failures or evidences of distress of major significance have occurred on sections of highways where surfacing depths were determined by this design procedure. In the few instances of minor distress, all were caused by the presence of sub-standard materials in the roadway section at a depth which did not provide the necessary cover as determined by this method of flexible pavement design.

The performance of pavements in the WASHO Road Test at Malad shows a fair degree of correlation between required surfacing depths and recommendations based on our design method. Although we do not now make any surfacing depth correction for the cohesion of the mat, the results at Malad indicate such could be used, particularly if shoulders are paved. Our design procedure is easily adaptable to such a modification.

The question of pavement deflection over resilient soils is not considered at present in our flexible pavement design. The importance of that phenomenon in the performance of bituminous pavements was clearly shown at the Malad road test. Considerable data have also been accumulated in California on deflection studies (2). The integration of deflection, or more properly resilience, with the stabilometer and swell pressure tests is necessary for a complete and rational analysis of the load-carrying ability of any subgrade soil. The inclusion of resilience tests in a routine laboratory design procedure will, however, require development of suitable testing machines and methods as well as cooperation among all highway and soils engineers in obtaining more data on the role of deflection in pavement service and durability.

ACKNOWLEDGMENTS

The graphs, data sheets, and other illustrations were prepared by H. E. Sandahl and W. A. Souers, Assistant Materials Engineers. Their work and help is gratefully acknowledged.

References

1. Hveem, F. N. and Carmany, R. M., The Factors Underlying the Rational Design of Pavements, Proceedings, Highway Research Board, Vol. 28, (1948).

2. Hveem, F. N., Pavement Deflections and Fatigue Failures, Bulletin 114 (1955) Highway Research Board.

Appendix A

Stabilometer Test Procedure

Samples of soil taken during the preliminary soil survey and representing materials which will be used in subgrades, are received in the Materials Laboratory. The samples are graded, and the portion passing the ¾-inch sieve is used for the stabilometer test. Five identical batches are then weighed out according to the grading of the ¾-inch minus material, each containing sufficient material to form a compacted test specimen 2½ inches high and 4 inches in diameter. Each batch is mixed with the same amount of water (approximately ½ to ⅔ the optimum moisture content) and the mixtures placed in individual plastic bags. The bags are closed with a rubber band and remain sealed overnight to allow the soil to "temper." Following the tempering the soils are mixed with more water and compacted in a 4-inch diameter steel mold by a triaxial insititue kneading compactor, Figure A. The quantity of water used in the mixing is such that it will give a compacted soil specimen from which water will be exuded by the application of a vertical load producing pressures between 100 and 600 psi on the specimen. The compactive effort consists of 40 blows at a foot pressure of 100 psi.

The compacted specimen is then placed on an exudation indicator called the Washington Visual Saturation Indicator, or the "peek-easy." This apparatus is merely a 1-inch thick piece of plexiglass mounted on a suitable framework. A pattern or target in the shape of a 4-inch circle with six equally spaced radii is scribed into a thin sheet of clear acetate which is placed on top of the plexiglass. A filter paper and a perforated 4½-

inch diameter disc made of bronze sheet are placed on top of the target. The perforations in the bronze disc consist of twenty-four ⅛-inch holes in the form of a circle 3¼ inches in diameter. A tilted mirror placed in the base of the framework allows the operator to view the piece of filter paper through the plexiglass and the scribed target on the acetate, Figure B.

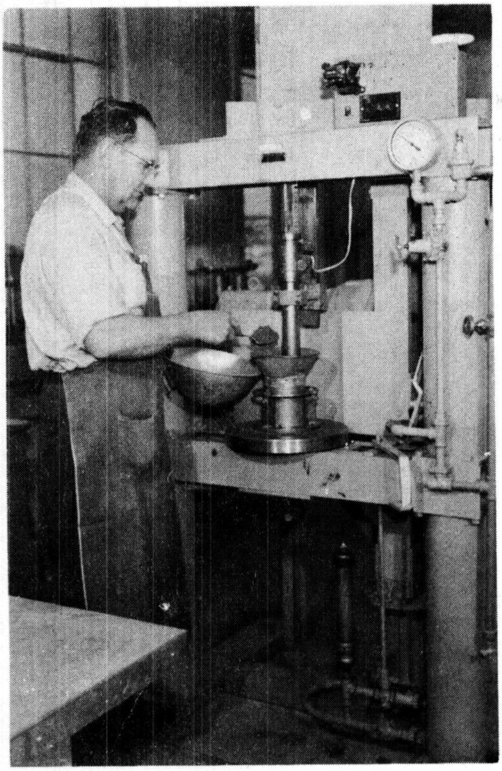


Figure A.

The compacted specimen in the steel mold is placed on the bronze disc and a vertical load applied to the soil. As the load increases, water is squeezed from the soil and travels through the perforations, moistening the filter paper in a circular pattern. The application of the vertical load is stopped when the circular pattern is continuous through ⅚ths of the circumference, Figure C. The unit pressure at which this occurs is called the exudation pressure. The range of exudation pressures considered satisfactory is 100 to 600 psi. A break-away sketch of the "peek-easy" is shown in Figure D.

Four specimens are compacted in a manner similar to that described previously at moisture contents which will produce four different exudation pressures within the specified range. This procedure produces test specimens which have densities comparable to those obtained in the roadway after construction and which also have physical properties such that resultant surfacing requirements agree with observed roadway performance of the material.

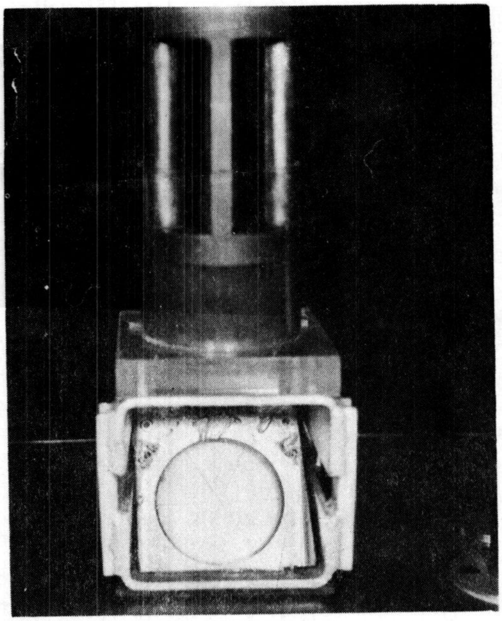


Figure B.

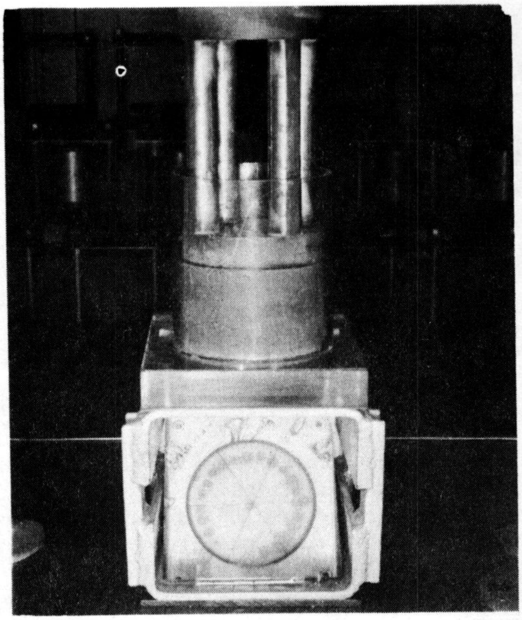


Figure C.

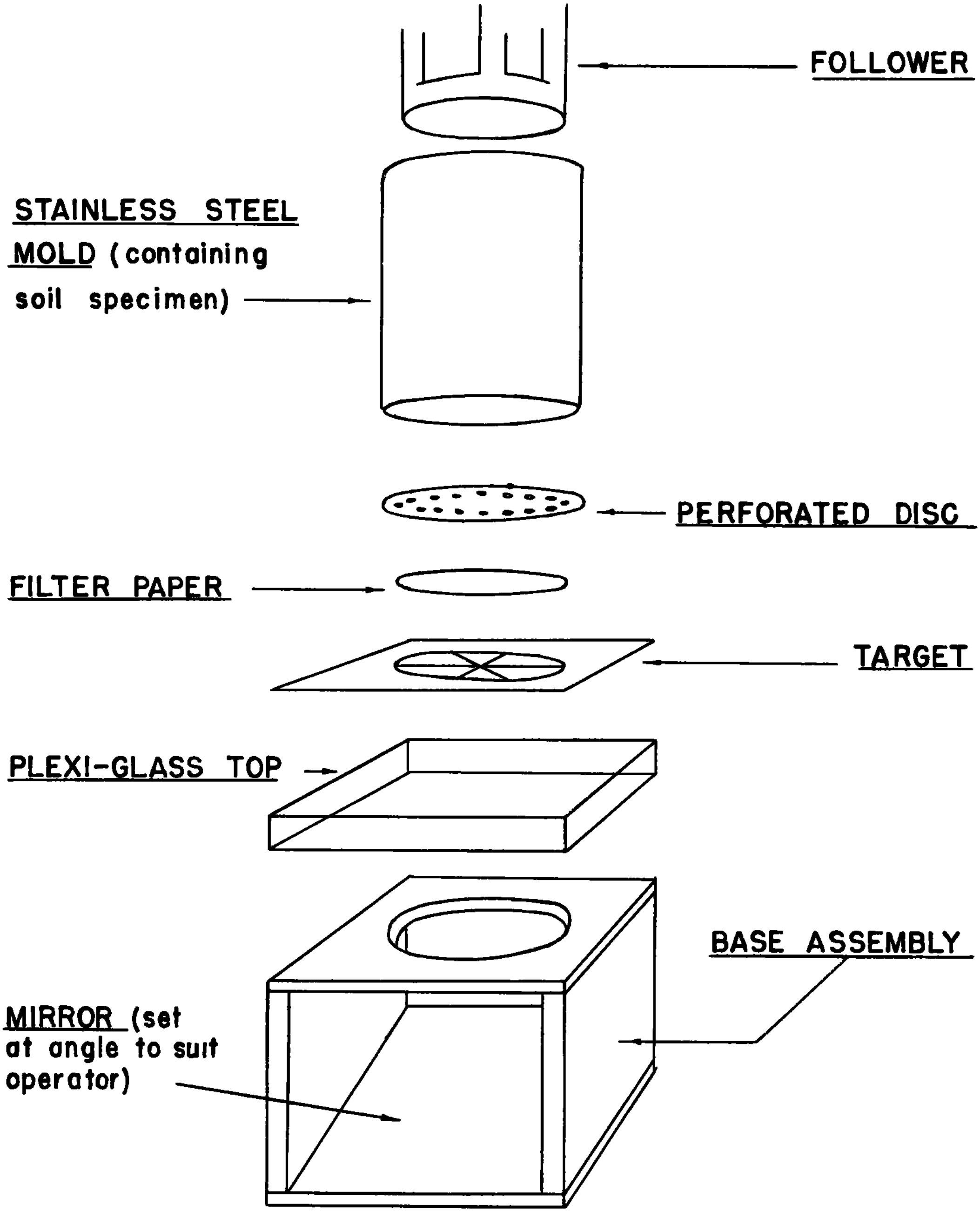


Figure D. Washington visual saturation indicator.

Following the exudation pressure determination, the test specimen, still in the steel mold, is placed in a swell pressure apparatus, Figure E. This device consists of an adjustable base on which the specimen is placed, a horizontal steel proving bar supported by two vertical posts, and a micrometer dial indicator for measuring the deflection of the proving bar. A perforated circular plate with a vertical stem is placed on top of the soil specimen and the height of the base adjusted to allow the stem to contact the horizontal proving bar. Water is placed on top of the soil specimens and allowed to remain there overnight. The swell pressure is measured by means of the bar deflection, the bar being calibrated so that a vertical deflection of 0.0001-inch is equivalent to a swell pressure of 0.04 psi.

After the swell pressure has been determined the soil specimen is tested in the Hveem stabilometer. The soil specimen is extruded from the mold into the body of the stabilometer and a lateral seating pressure of 5 psi applied by means of the displacement pump on the stabilometer. A vertical load is then applied to the specimen at a strain rate of 0.05 inches per minute and lateral pressure readings are taken at vertical loads of 500, 1,000, and 2,000 lb. The vertical load is then reduced to 1,000 lb and the platen of the testing machine maintained at this position (not necessarily this load) while the lateral pressure is reduced to 5 psi by means of the displacement pump. The number of turns necessary to increase this lateral pressure from 5 psi to 100 psi is then measured and the figure recorded as the displacement, "D." Figure F shows the stabilometer test in progress.

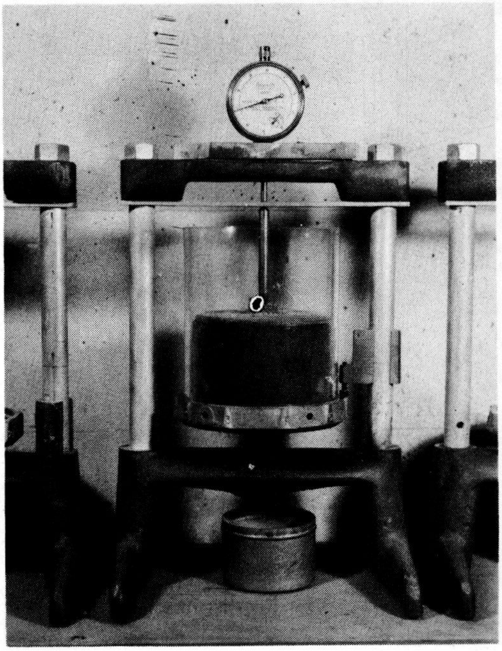


Figure E.

The stabilometer "R" value is calculated from the above data according to the following formula:

$$R = 100 - \frac{100}{\frac{2.5}{D}\left(\frac{P_V}{P_h} - 1\right) + 1}$$

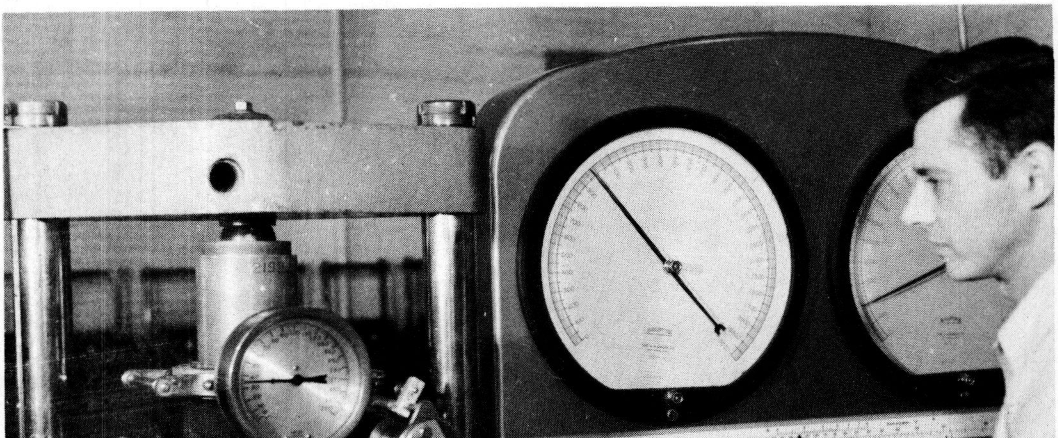


Figure F.

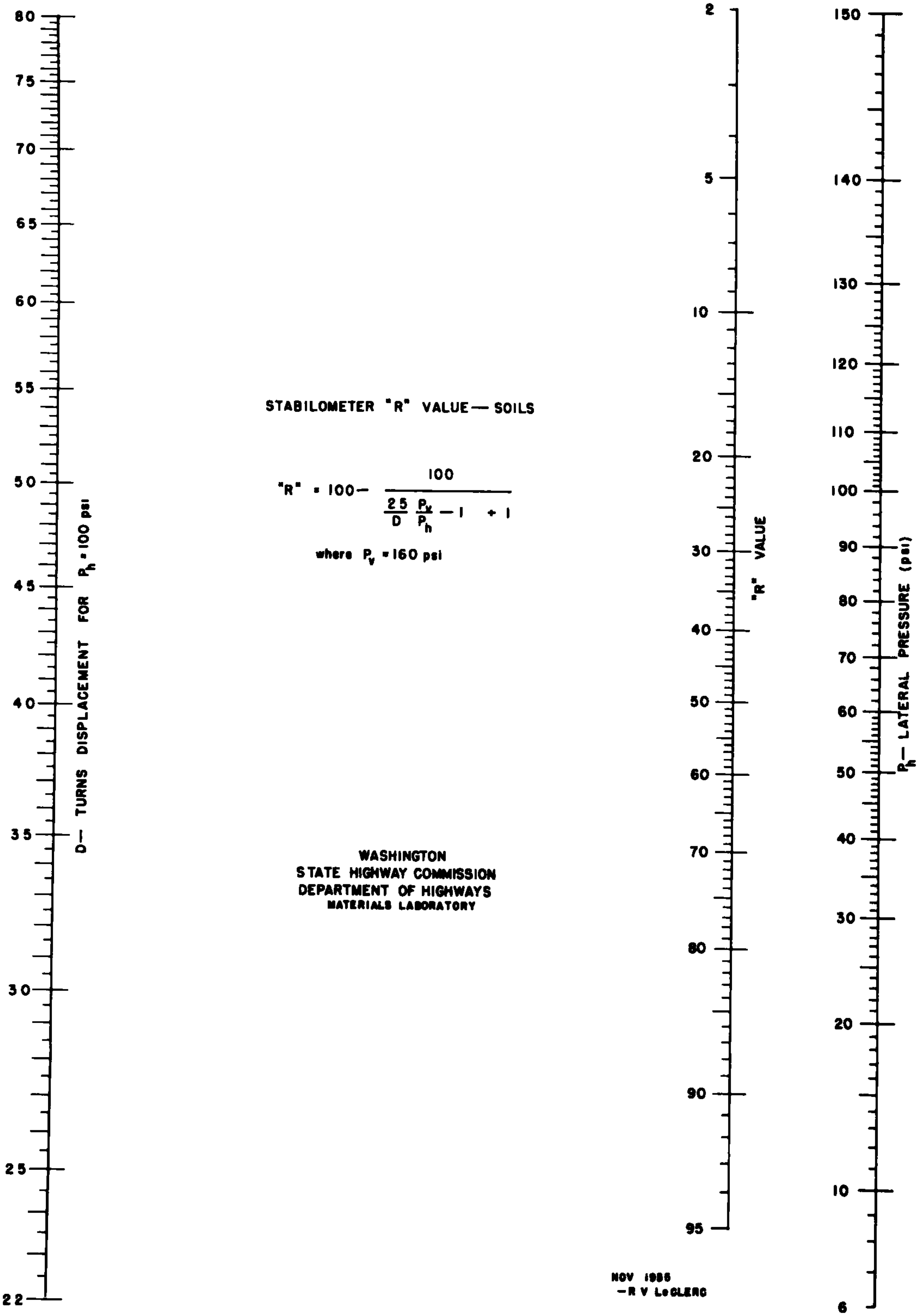

STABILOMETER "R" VALUE — SOILS
"R" = 100 − 100 / (25/D (Pv/Ph − 1) + 1)
where Pv = 160 psi
WASHINGTON
STATE HIGHWAY COMMISSION
DEPARTMENT OF HIGHWAYS
MATERIALS LABORATORY
D — TURNS DISPLACEMENT FOR Ph = 100 psi
"R" VALUE
Ph — LATERAL PRESSURE (psi)
—R V LeCLERC

Figure G.

Where D = Displacement
P_v = Vertical pressure (160 psi at 2,000 lb)
P_h = Horizontal or lateral pressure at 2,000 lb total load.

Solution of this equation is accomplished by means of a nomograph, Figure G, or a large slide rule constructed for this purpose and mounted on the testing machine used in the test.

Appendix B

Field Soil Report Form

In an effort to aid the district soils engineers in preparing their field reports and to insure adequate coverage of all pertinent points, the attached outline is suggested as a form for field soils reports. Some of the topics will not be applicable in many cases, but some jobs conceivably could require a coverage of all of the items shown.

In compiling the field report, it is recommended that all topic headings be listed and only those pertinent to the particular job be discussed. Under those topics requiring no discussion, a short statement to that effect with reasons therefor should be sufficient.

FIELD SOIL REPORT - TOPICS

I. General

a. Description of contemplated project construction - plan views of project may be incorporated in soil profile, if necessary, to aid in description. Photographs may also be attached.
b. Climatic conditions - amount of rainfall, local frost conditions, etc.
c. Traffic conditions - type of traffic, contemplated volume increases, etc, - estimate of access connection traffic.
d. If resurfacing construction, notes on possibility of grade changes.
e. Control section involved.
f. Status of project - when scheduled for construction, etc.

II. Geology and Physiography

a. General topographic features, if pertinent.
b. History and description of condition of nearby roadway or other structures, if similar conditions prevail.

III. Soils

a. Brief summarized description of soil profile for the project.
b. Limits within which soils represented by submitted samples will govern surfacing design, if possible.
c. Comments on soils of questionable stability and general evaluation of soils in the proposed roadway.
d. If resurfacing construction, some relation between pavement condition and supporting soils.

IV. Fill Foundations

a. Comments relative to stability of foundations and dimensions of proposed fills.
b. Brief description of foundation profile.
c. Brief description of investigational work and findings, if such work is necessary.
d. Location of water table.

V. Slope Stability

a. Description of conditions where slope stability has required investigation, and description of investigational work.

b. Comments on stable slopes in similar material, if any.
c. Slope erosion possibilities.
d. Potential slide conditions.

VI. Drainage and Water Conditions

a. Comments on special drainage features which might reflect on soil stability.
b. Location of water table where pertinent.

VII. Materials Available

a. Source of material to be used over subgrade (selected roadway borrow, cement treated base aggregate, sand drain backfill, etc).
b. Any special materials having to do with soils problems.

VIII. Special Features

a. Stabilization courses, special soils blending, etc.
b. Feasibility of above.
c. Existence of solid rock, if not covered elsewhere.

IX. Recommendations

a. Optimum use of soil materials through selective placement, order of construction, etc.
b. Recommendations pertaining to any one topic may be included under that topic if considered more appropriate.

HRB:OR-11

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