A Tentative Flexible Pavement Design Method for Florida

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The current results of a study being made to develop a design procedure for determining the thickness of the layers of a flexible pavement structure to carry a specified predicted traffic load are described. The objective was to develop a correlation between bearing values of materials, layer thicknesses, traffic loads, and pavement performance.

The approach was to first select representative pavements in rural locations with 10-year performance records and observed condition ratings of poor to very good. Actual field conditions were determined at each test site, including moisture content, density and field bearing value of each layer, transverse and longitudinal profiles, and evlauation of surface cracking. Field densities and moisture contents were compared with laboratory tests for optimum moisture and density.

A rating system was developed which indicated by a single number the condition of the pavement in regard to deviation of longitudinal profile, depth of rutting, and degree of surface cracking. The resulting condition index was adjusted for traffic volume and converted to a service rating. A service rating of 60 was considered to be a realistic value for the dividing line between poor and good performance.

Thickness requirements were developed using service rating, equivalent wheel load data, and a modified California bearing test. Design curves that predict all unsatisfactory pavement performance with a minimum of over-design are presented.

IN 1955, the State Road Department of Florida initiated a study of its flexible pavement design method. The major purpose of the study was to evaluate the design criteria and the performance of flexible pavements constructed in Florida. At this time, the Department used an empirical method of design which generally resulted in a standard section design accompanied by minimum Florida bearing values. This method has been described in detail in a recent report (1).

In initiating the study, numerous design methods were reviewed and it was found that the California bearing ratio (CBR) design method or a modification of this method was the most widely used and accepted. In addition, extensive research had been performed in conjunction with this method by many agencies and principally the Corps of Engineers. Inasmuch as valuable information and experience was available on the CBR method, the state considered using this basic method if it was found that a new design method was warranted. Research was also planned to include work, at a later date, with plate tests and layered theory.

The first field and laboratory study was undertaken in 1957. The major findings and developments of this study were reported (1, 2) and are briefly as follows:

1. Test sites were selected using selective sampling techniques. The sites were limited to sections of rural highways constructed in 1947 or 1948, with "observed conditions" ratings of poor through very good. Seventy-six test sites were selected of which 23 sites were investigated in 1957. In 1959 28 additional sites were studied (Fig. 1).

2. A rating system was developed which indicated by a single number the condition of the road section relative to any other section; the rating system considered classes of cracking, deviation of longitudinal profile, and average depth of rutting. A "condition index" number resulted when the factors were weighted and expressed as a product (2, 3) (Fig. 2).

3. The condition index was adjusted for traffic according to a family of curves developed from the U.S. Bureau of Public Roads (4). An adjustment was derived for conditions existing in Florida, and the original Bureau of Public Roads' equation modified. The condition index when adjusted for traffic was denoted as service rating.

4. Field and laboratory strength tests performed on the base, subbase and subgrade were related to service rating. It was found that:

(a) The Florida bearing design methods did not correlate with performance.

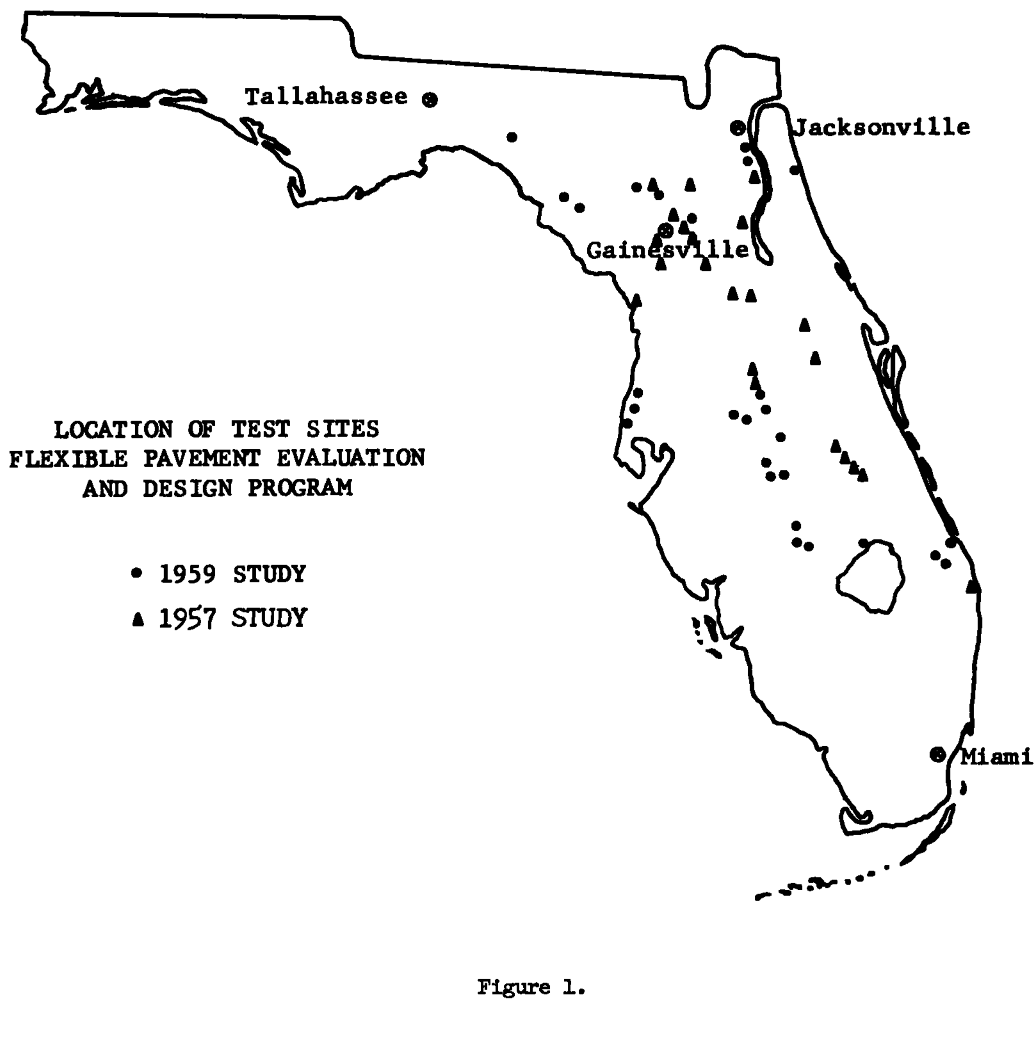


Figure 1.

(b) The California bearing ratio design method (5) did correlate with performance when a 12,000-lb wheel load design curve was used for all the test sections which had an average ADT of 1,850, and a service rating of 60 was used as the dividing line of good and poor performance.

The use of a service rating of 60 was considered to be realistic when compared to visual inspection and engineering judgment.

During 1958, field and laboratory studies were continued and additional data obtained to further evaluate performance and develop a design method for Florida. The investigation dealt primarily with flexible pavements having limerock as a base material. However, a very limited study was made of sand clay and other base materials.

FORMULA FOR DETERMINING CONDITION INDEXES

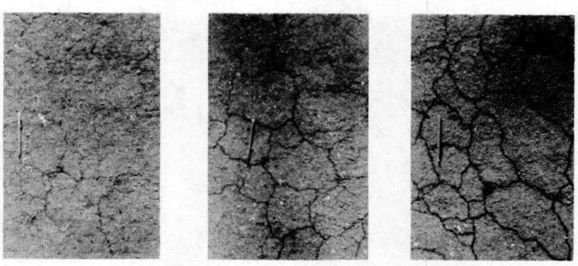
$$C = 100 \left[\frac{1 + \left(1 - \frac{b_1}{c_1}\right)\left(1 - \frac{b_2}{c_2}\right)\left(1 - \frac{b_3}{c_3}\right)\left(1 - \frac{b_4}{c_4}\right)}{2} \right] \left[1 - \frac{b_5}{c_5} \right] \left[1 - \frac{b_6}{c_6} \right]$$

b_1 = Area of Class 1(A) cracking divided by total area.	$c_1 = 10$
b_2 = Area of Class 2(B) cracking divided by total area.	$c_2 = 4$
b_3 = Area of Class 3(C) cracking divided by total area.	$c_3 = 2$
b_4 = Area of Class 4(D) cracking divided by total area.	$c_4 = 1$
b_5 = Standard deviation of longitudinal profile of original pavement, inches.	c_5 = 1 inch
b_6 = Average depth of rutting in wheel paths of original pavement, inches.	c_6 = 2.25 inches

Substituting the values listed above in Equation 1 with n = 6, the Condition Index for flexible pavement is computed by:

$$C = 100 \left[\frac{1 + \left(1 - \frac{b_1}{10}\right)\left(1 - \frac{b_2}{4}\right)\left(1 - \frac{b_3}{2}\right)\left(1 - b_4\right)}{2} \right] \left[1 - b_5 \right] \left[1 - \frac{b_6}{2.25} \right]$$

Note: In patched or replaced areas the last recorded values of cracking, longitudinal profile, or rutting are to be used.



A B C D

(A) Class 1 cracking - fine cracks with no well defined pattern.
(B) Class 2 cracking - fine cracks with a grid-like pattern.
(C) Class 3 cracking - similar to Class 2 with widening of the cracks and some spalling along the edges.
(D) Class 4 cracking - progression of Class 3 cracking with pronounced widening of the cracks and separation of the resulting segments into individual loose pieces.

Figure 2.

All of the test sections in this study were 11 yr old and were of the same group selected for study in 1957. Selection of the test sites was discussed in detail in a previous report (2). Table 1 gives the number of test sites, class of traffic, and the range of average ADT for the 11-yr period.

Field data were obtained as in the 1957 study. However, additional field CBR tests were made. These were principally on the base and subgrade materials. The field testing was essentially as follows:

TABLE 1

TRAFFIC DATA 1959 STUDY

Traffic Class	11 Years Total Range of Traffic	Number of Test Sites	Range of Actual Average ADT (2 lane)
1	$0 - 1 \times 10^6$	1	125
2	$1 - 2 \times 10^6$	2	396 - 399
3	$2 - 3 \times 10^6$	2	725 - 725
4	$3 - 6 \times 10^6$	4	800 - 1,190
5	$6 - 9 \times 10^6$	6	1,535 - 2,080
6	$9 - 12 \times 10^6$	6	2,380 - 2,812
7	Over 12×10^6	7	3,220 - 4,568

Note: Average of System 2,070.

1. Profiles were established, both longitudinal and transverse (Fig. 3);
2. Pavement surfaces were evaluated for class of cracking;
3. Field CBR tests were made on all the layers encountered (Fig. 4);
4. Field density and field moisture tests were made on all layers, in and between wheel paths and on the shoulders; and
5. Samples were obtained for laboratory testing.

PRELIMINARY FINDINGS

As a result of the field and laboratory studies, preliminary correlations between density, moisture and rutting were made. The important findings and pertinent comments are as follows:

1. The average field density of the base material was about 95 percent of the maximum laboratory density determined by modified AASHO methods.
2. The average field density of the subbase material was generally equal to (100 percent) and sometimes greater than maximum laboratory density.
3. The field moisture content of the base material (limerock) was generally equal

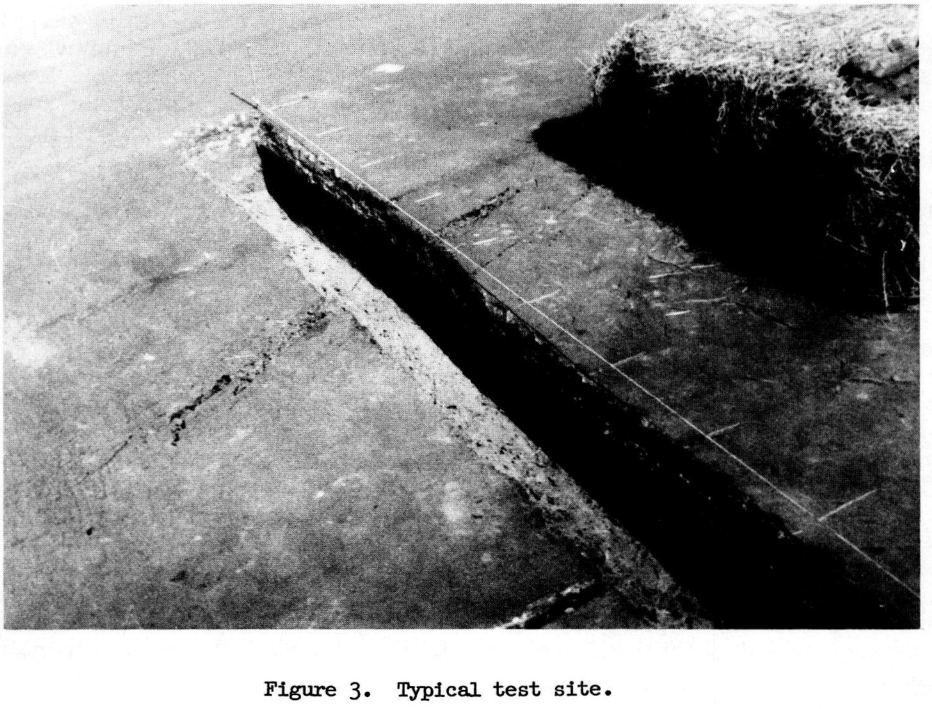


Figure 3. Typical test site.

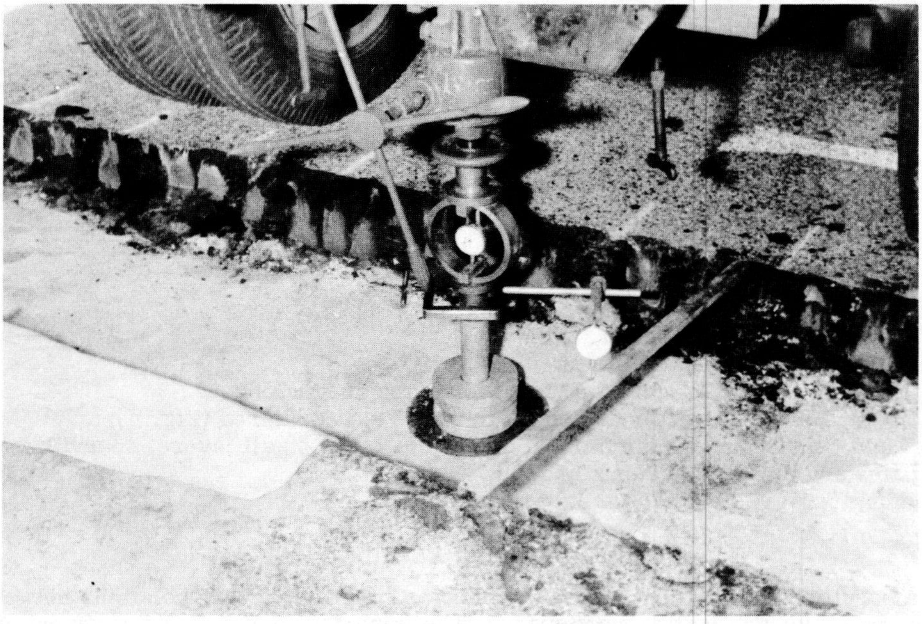


Figure 4. Field CBR test and density tests.

to the optimum laboratory moisture content as determined by modified AASHO methods. This same condition existed in the 1957 study, indicating that during the period of field testing, the base material did not reach a moisture content higher than optimum and was less than the moisture contents obtained when testing two-day capillary soaked CBR test specimens. Future field moisture content studies will be made during the wettest months to determine whether or not the field moisture content increases.

4. The field moisture content of the subbase material was generally equal to or less than the optimum laboratory moisture content. The field moisture was typically about 60 percent of laboratory optimum. This relationship was found to exist in previous studies and again indicates the need for future field moisture content studies to determine the most realistic moisture content for testing CBR specimens.

5. The field density beneath the outer wheel path was generally equal to the field density beneath the inner wheel path. This held true for both base and subbase.

6. The field moisture content beneath the outer wheel path was generally equal to the field moisture content beneath the inner wheel path. This was true again for both base and subbase.

7. The rutting in the outer wheel path was greater (50 to 150 percent) than the rutting in the inner wheel path. This was expected to occur on this system with roads constructed with stabilized soil shoulders only. Because the density and moisture showed no appreciable variation in the inner or outer wheel paths, it appears that the increased rutting in the outer wheel path may be attributed to shear-strain displacement. The shear resistance of the shoulder material and underlying native soil is undoubtedly less than the base and subbase and thus would lead to greater shear-strain displacement. Some evidence of displacement adjacent to the pavement section was noticed when the test sites were trenched. Extending the base course and surface would reduce outer-wheel-path rutting and essentially make the outer wheel path similar to the inner wheel path. The rutting beneath the inner wheel path was undoubtedly due to the compression of the base and subbase and to shear-strain displacement ("Special Shoulder Treatment to Alleviate Wheel Path Rutting," Research Bulletin 21, State Road Department of Florida).

CBR METHOD OF DESIGN

Major emphasis of the study was placed on investigating the CBR method of design or a necessary modification of this method. It was important to determine again if performance, service rating, and condition index could be related to an established design method or if modification of the method or design criteria would be necessary.

The CBR method studied was than originally proposed by O. J. Porter (5). The method has since been adopted in the original and revised forms by numerous states and the Corps of Engineers has done extensive work in reviewing and developing this method.

The CBR method basically consists of evaluating the bearing strength of a compacted laboratory specimen or field specimen, by penetrating the sample with a circular piston of 3 sq in. The stress obtained at 0.10-in. penetration is compared to a standard stress (1,000 psi). Prior to testing, the sample is normally subjected to soaking. The Corps of Engineers requires four days of soaking in order to simulate the worst possible field conditions. Other agencies require the same or less drastic soaking conditions. The State of Florida has found that a two-day capillary soak period would approximate most of the worst field conditions but that no soaking appears to be more typical of the average field conditions (1, 6). Both conditions of testing were used and studied by Florida and a complete testing procedure is in preparation. The desirability and advisability of soaking test specimens is discussed later.

EVALUATION OF PAVEMENT SECTION DESIGN

The flexible pavement sections investigated were typical of those constructed in 1947 and since. The over-all thickness of wearing surface, base, and subbase was generally greater than 17 in. and in most sections had a thickness of 19 in. The wearing surface varied generally from 0.5 to 2 in. in thickness, the base generally from 6 to 8 in., and the subbase from 10 to 12 in. The service ratings varied from 28 to 75. This variation included very poor through good performance. Excellent performance was not encountered on any of the test sections.

As mentioned, previous studies indicated that the original CBR curve for medium heavy traffic would generally define the thickness requirements and the strength index of flexible pavements constructed in Florida which have been subjected to 11 yr of traffic. A service rating of 60 divided good and poor performance, based on a limited number of test sections, the average ADT of the sections being 1,850 v.p.d.

The first analysis of the most recent survey dealt with the development of design curves based on CBR and service rating, service rating being the condition index adjusted for traffic volume. As a result of this analysis, a family of design curves was developed for thickness requirements as related to volume.

It was suspected that volume and equivalent wheel loads (EWL) were directly related in the State of Florida. The analysis on volume alone would be fairly reliable if volume and EWL were related.

The second analysis considered equivalent wheel loads (EWL) and condition index. Wheel load data were obtained (7) in July 1959, which permitted the analysis to be made.

Preliminary examination of the CBR data, the general thickness requirements, and the actual constructed thickness indicated that most failures could be attributed to either a weak base material, when wet, or an inadequate thickness of base for a weak (low CBR) subbase. Combined thicknesses of base and subbase appeared sufficient in all but one case to prevent failure of the subgrade. Analysis of the flexible pavement sections presented was made of the wearing surface, base material, subbase material, and subgrade.

Wearing Surface

The wearing surfaces encountered were of three major types: (1) surface treatment, (2) plant mix retread, and (3) asphaltic concrete. No major study was made of the wearing surface, however, it was possible to draw some general conclusions related to wearing surface.

Both surface treatments and asphaltic concrete surfacing gave satisfactory service although a somewhat higher percentage of surface-treated sections failed.

Surface treatments if properly maintained should be satisfactory. Lack of maintenance would undoubtedly lead to surface moisture penetration and weakening of the base. For thin surfacing, shear stress beneath the loaded area would definitely penetrate into the base material. This would warrant excellent base material of high shear resistance. The use of borderline, low-CBR base material should be avoided when the surface is of the surface-treatment type.

TABLE 2

WEARING SURFACE, MINIMUM REQUIREMENTS[1]

2 Lane ADT	Type of Surface
300 - 1,000	Double surface treatment
1,000 - 3,000	Triple surface treatment
3,000 - 4,000	Asphaltic concrete, 2½ in.
4,000 - 6,000	Asphaltic concrete, 3 in.
Over 6,000	Asphaltic concrete, 3½ in.

[1] See Ref. (1).

Asphaltic concrete surface sections gave good service and generally were from 1.5 to 2.0 in. thick. Available information indicates the use of 2 to 3 in. as being desirable. The use of asphaltic concrete surfacing for roads with a predicted ADT of 3,000 v.p.d. or greater seems advisable and was found to exist on the sections studied. Table 2 gives the minimum wearing-surface requirements.

Base Course

The major base-course material studied was limerock. This is the most common material used in the state, although sand clay, clay, and shell mixtures are used as base materials. The main purpose of the base course is to distribute the wheel load to the underlying layer. It should not compress excessively, should withstand the shear stress imposed by the wheel load, and should be stable under all degrees of field moisture. Hard limerock, compacted to high density, meets the requirements of good to excellent base material.

A minimum laboratory four-day-soak CBR of 80 and in some cases as low as 60, at 95 percent maximum density, is required by many agencies. Complete submergence of the specimen and a four-day soak period is a severe test. The soak test performed by the State of Florida is not complete submergence, but a two-day capillary soaking with a head of water equal to the height of the specimen. This may also be considered somewhat severe inasmuch as the resulting moisture content of the test specimen is generally slightly greater than the field moisture encountered to date. It is expected, however, that future field moisture data will show that the two-day capillary soak period is quite realistic for Florida soils and climate.

The minimum field CBR value normally required for base materials is 80. Again moisture content is extremely important when obtaining the minimum bearing value in the field. The field tests performed in this study were run between 8 and 12 percent field moisture and may not be the maximum values; however, it is certain that they are fairly realistic because the base study performed earlier (6) gave similar results. The general range of CBR values obtained is given in Table 3 (a). Table 3 (b) presents the range of CBR values obtained as a result of earlier base course material studies (6).

The data in Table 3 show that both the CBR at optimum and the field CBR met the requirements for good base material. Some soak CBR values are low; however, these samples were tested at moisture contents slightly greater than those occurring in the field. For analysis it was assumed that the base material was sufficiently strong to prevent base failure where the soak CBR was greater than 40.

The fact that some limerocks lose strength rapidly with soaking cannot be overlooked. Poor performance did occur in some test sites where the CBR (soak) was less than 40. A combination of poor surface treatment and a soak CBR of less than 40 was accompanied by a low service rating in two test locations and it is probable that poor base material did contribute to the failure of the test section. For the same test locations, however, the subbase strength or the base thickness was inadequate, which would also lead to failure. It is, therefore, difficult to attribute all of the poor performance to poor base material. No test locations were encountered where adequate thickness of poor

TABLE 3

RANGE OF CBR VALUES AT 0.10-IN. PENETRATION

CBR Laboratory (Opt. Moist.)[1]	CBR Laboratory (2 Day Soak)	CBR (Field)
	(a) 1959 Study	
Limerock		
60 to 150	30 to 130	90 to 200
	(b) 1958 Study	
Ocala limerock		
31 to 175	27 to 140	60 to 204
Miami oolite		
45 to 228	30 to 145	95 to 290
Sand-clay		
10 to 75	15 to 100[2]	23 to 192
Shell		
55 to 65	38 to 58	35 to 124
Shell-sand		
15 to 90	15 to 90	34 to 130

[1] CBR Laboratory: optimum moisture or optimum as used in this report pertains to CBR test samples compacted at optimum moisture and maximum density obtained by compacting five equal layers at 55 blows per layer with a 10-lb hammer and 18-in. drop in a CBR mold having a volume of about 0.10 cu ft.

[2] Indicates drying of surface of sample. Corrected in later tests (9).

base material existed; nevertheless, it appears that all base materials should have a definite minimum soak CBR of 40 and a desirable lower limit of 60. It also is suggested that where poor base material, soak CBR 40 to 60, must be used, additional thickness of wearing surface should be used. Specific values of CBR for different classes of traffic are noted later.

Subbase Course

Preliminary examination indicated that the low service ratings could be attributed to low strength of the subbase or inadequate thickness of base. Both of these terms may be used interchangeably because when designing the flexible pavement, base material may replace subbase. However, at any given depth beneath the surface, the subbase, if used, must have a minimum bearing value.

The technique used to correlate service rating and design curve was to plot actual thickness versus required thickness for a CBR value and note for each test site the service rating. Sites where the actual thickness is greater than the required thickness should have high service ratings. The opposite should result in low service ratings. The line of equality (actual thickness equals required thickness) should divide the good performance sites from the poor.

Actual thickness was obtained from field measurements, required thickness from the medium heavy traffic design curve. Having plotted the data it was possible to adjust the required thickness so that a better correlation resulted. This was done for all studies where desirable.

Another method of evaluating the data is to plot actual thickness against CBR, on a semi-logarithmic plot. When service rating is plotted a curve may be drawn which best fits the data and this then results in the design curve. Either method would obtain essentially the same end result.

The use of service rating numbers permits a fairly accurate analysis of the data. The original analysis included actual service-rating numbers. A later report grouped service rating as follows:

Grouping of Service Rating (SR)

A. Test sections—Service rating 30 to 59—Definitely poor performance
B. Test sections—Service rating 60 to 65—Questionable performance
C. Test sections—Service rating 66 to 75—Definitely good performance

Analyses were made of the field CBR, laboratory CBR at optimum moisture, and laboratory CBR after two-day capillary soaking. The subbase materials tested were principally A-3 and A-2-4 soils. The following conclusions resulted: Considering the field CBR, the use of the original California curve would predict about 38 percent of the failures. Adjusting the design curve by adding 0.5 in. to the required thickness would predict 50 percent of the failures. Adding about 4 in. would predict all failures.

Considering the laboratory CBR at optimum the use of the original California curve would predict about 56 percent of the failures. Adjusting the design curve by adding 0.1 in. would predict 67 percent of the failures. The use of 0.1 in. is not intended to illustrate accuracy of a design method but merely to show the small magnitude of adjustment required. Adjusting the design curve by adding about 2 in. would predict all failures.

Considering the laboratory CBR when a soaked specimen was tested, the use of the original design curve would predict 89 percent of the failures. Adding about 0.5 in. would predict all failures. The use of the soak CBR and the adjusted design curve would predict all failures but would result in slight overdesign for 8 out of 28 test sites.

As a result of the subbase study it was found that all of the failures could be attributed to inadequate thickness of base or insufficient bearing strength of the subbase. This conclusion could be drawn inasmuch as the actual total thickness of surface, base and subbase exceeded the required thickness, as obtained from design curves for wheel loads as high as 20,000 lb. Actual findings are noted in the discussion of the subgrade.

The CBR of a soaked sample would certainly be the best criterion for predicting failures but a maximum number of overdesigned sections would result. However, it appears that the use of the soak CBR test is desirable as a control test until a field moisture content study is completed which will cover all sections of the state and includes the various wet seasons. The use of the soak CBR for design will give general assurance of satisfactory performance and should be used until additional moisture data are obtained.

Subgrade

Flexible pavements distribute wheel loads from the surface through the underlying layers to the subgrade. The normal stress and maximum shear stress are at or near the surface and diminish with depth. Inasmuch as this type of stress distribution exists, it is logical to develop a pavement section having layers which decrease in strength with depth. This could be simply stated in terms of CBR—the CBR of the base should be greater than the CBR of the subbase which in turn should be greater than the CBR of the subgrade. This may be termed a normal flexible pavement section. The review of the subgrade was concerned principally with normal pavement sections. The subgrade materials were principally A-3 soils although a few A-2-4 soils were encountered.

Because the failures could be attributed to poor subbase or inadequate thickness of base, analysis of the subgrade or combined thickness of surface, base, and subbase should result in all sections having adequate required thickness. This was found to be true when actual total thickness was compared to required thickness.

Analysis based on the field CBR and service rating indicated that all sections had adequate thickness of material above the subgrade. A reduction of the actual design thickness of about 3 in. would give a better correlation.

Analysis based on the CBR at optimum moisture also indicated adequate thickness above the subgrade. A reduction of the actual design thickness of about 1 in. would improve the correlation.

Analysis based on the CBR when a soak specimen was tested indicated adequate thickness and very little reduction of design thickness was indicated.

As a result of the subgrade study it was found that sufficient total thickness of surface, base, and subbase existed in all test sites. The use of the original CBR design curve, and the two-day capillary soak test, showed the best correlation.

Design Curve

Figure 5 shows the corrections resulting from the various studies, the general limits of the study, and the resulting design curves. Additions and reductions noted previously are shown, as well as the CBR ranges investigated.

The design curve for the average of all test sites based on service rating is shown in Figure 6. This curve is for an ADT of 2,070 and a service rating of 60. The two-day capillary soak CBR, which was found to be the most realistic for predicting performance, is used. All flexible pavement sections designed on this basis could be expected to have a service rating of 60 or more for an ADT of 2,070 after 11 yr of traffic. The condition index would also be equal to 60 for this traffic volume.

Design Curves Related to Volume

Service rating was related to condition index and volume by

$$R = c + \frac{100\,c - c^2 \left(\log \frac{T}{T_s}\right)}{K} \tag{1}$$

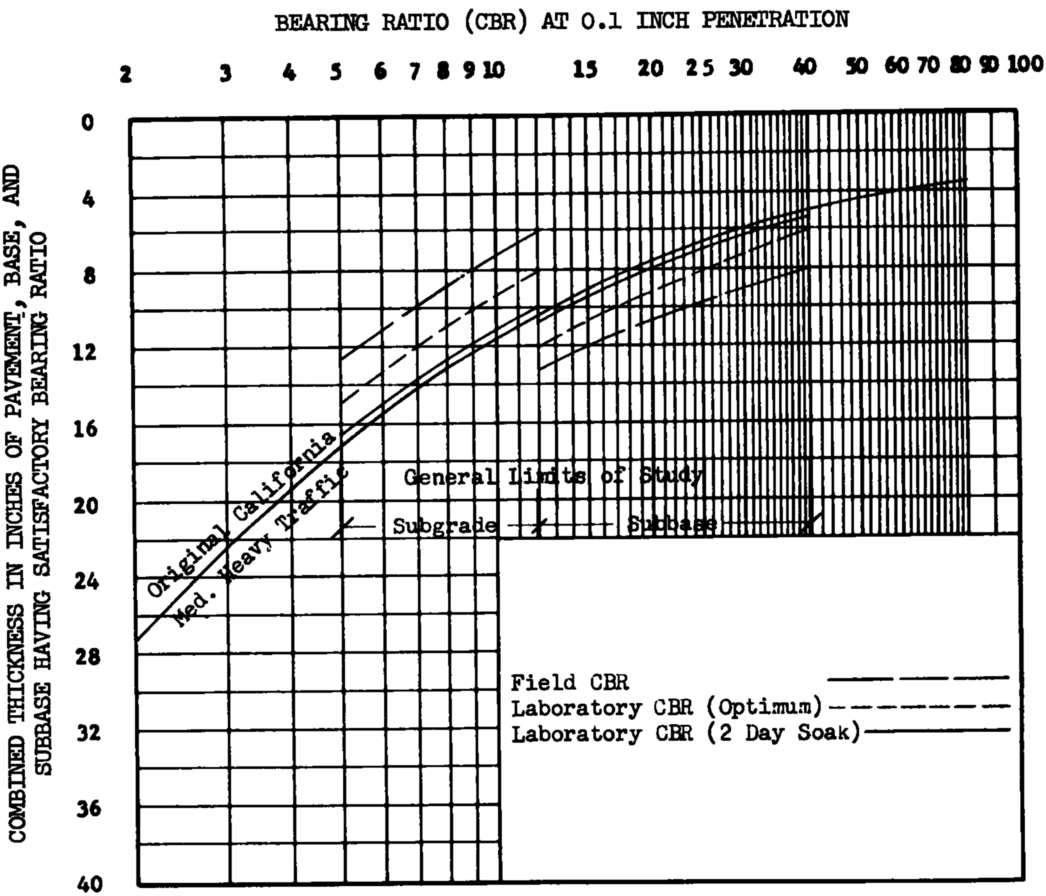


Figure 5. Adjusted design curves based on service rating for the average volume of the system.

and

$$K = \frac{100\,c - c^2}{50\,(\log T_s - \log T_m)} \tag{2}$$

in which

- R = service rating;
- c = condition index;
- T = ADT on section;
- T_s = ADT on system; and
- T_m = minimum ADT = 50.

For a condition index of 60 and an average ADT of 2,070, K = 29.7 and

$$R = c + \frac{100\,c - c^2\,(\log \frac{T}{T_s})}{29.7} \tag{3}$$

Examination of service-rating and thickness data indicated that a definite trend resulted. When the actual thickness was greater than the required thickness the service rating increased. When the actual thickness was less than the required thickness the service rating decreased. Inspection of these trends led to the developments of service rating limits (Fig. 7a). A service-rating value of 60 is the dividing line between good and poor performance and has an ADT of 2,070. The condition index for this volume is also 60. Using Eq. 3 volumes were computed for the service-rating limits and a condition index of 60. Having calculated the volume, a family of design curves resulted.

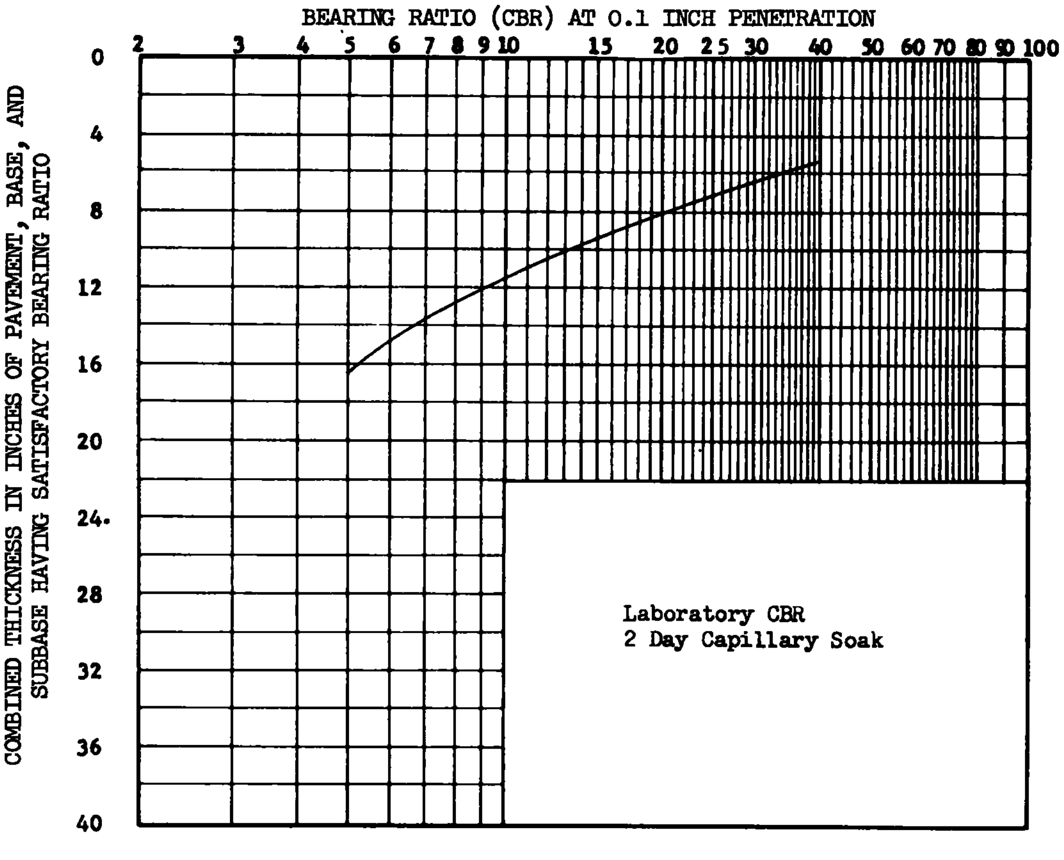


Figure 6. Design curve based on service rating for the average volume of the system (Volume 2,070).

The design curve for a volume of 2,070 and a condition index of 60 would require a service rating of 60 and would be the original curve developed. A design curve for a volume of 6,600 and a condition index of 60 would require a service rating of 75. The service-rating limit of 75 developed from the data would require an additional thickness of 4 in. A design curve for a volume of 440 and a condition index of 60 would require a service rating of 40. This would result in a reduction of the thickness by 3 in. Figure 7b is a graph of required thickness and corresponding volume for a condition index of 60.

Using the information in Figures 7a and 7b, a family of design curves was prepared (Fig. 8).

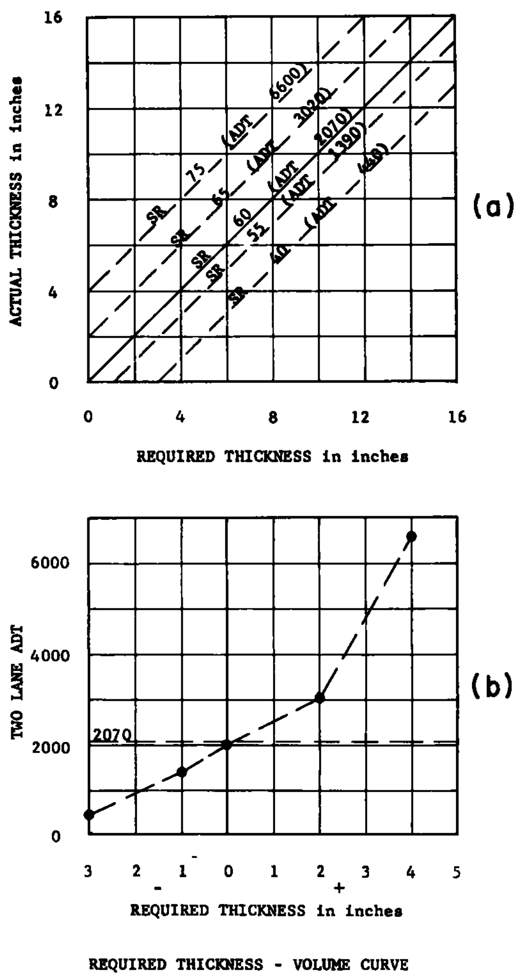


Figure 7. Service rating—volume curves for a condition index of 60.

Design Curves Related to Equivalent Wheel Load

Preliminary work in 1957 indicated that equivalent wheel loads may be directly related to volume. Since then the Traffic and Planning Division has performed an analysis relating volume, a lane ADT, and equivalent wheel loads, and the results have been used for the following correlation. Table 4 gives the important data used in the development of the design curves based on EWL and includes such information as test site, class of traffic, average ADT, total EWL, condition index, and service rating. The data indicate that EWL is almost directly related to volume and that class of traffic generally defines the range of EWL.

For the analysis of equivalent wheel loads and the development of design curves three ranges of EWL were studied. These were as follows:

1. EWL range 5 to 15 x 10^6; total for 11 yr, 2 lanes.
2. EWL range 19 to 33 x 10^6; total for 11 yr, 2 lanes.
3. EWL range 36 to 60 x 10^6; total for 11 yr, 2 lanes.

The 19 to 33 x 10^6 range of wheel loads included the maximum number of test sites—thirteen—and had an average ADT of 2,321 v.p.d. The average volume of the system was 2,070 v.p.d.

The technique of analysis was to plot actual thickness versus CBR on a semi-logarithmic plot and note the condition index. EWL includes the effect of volume, therefore service rating was not used for correlation. Using this presentation, a design curve which best fitted the data could be drawn directly. No adjustment would be necessary as when analyzing actual and required thickness. A similar analysis was used by Kentucky (8).

For each of the three EWL ranges, design curves have been developed for the field CBR, the laboratory CBR at optimum, and the laboratory CBR when tested after soaking.

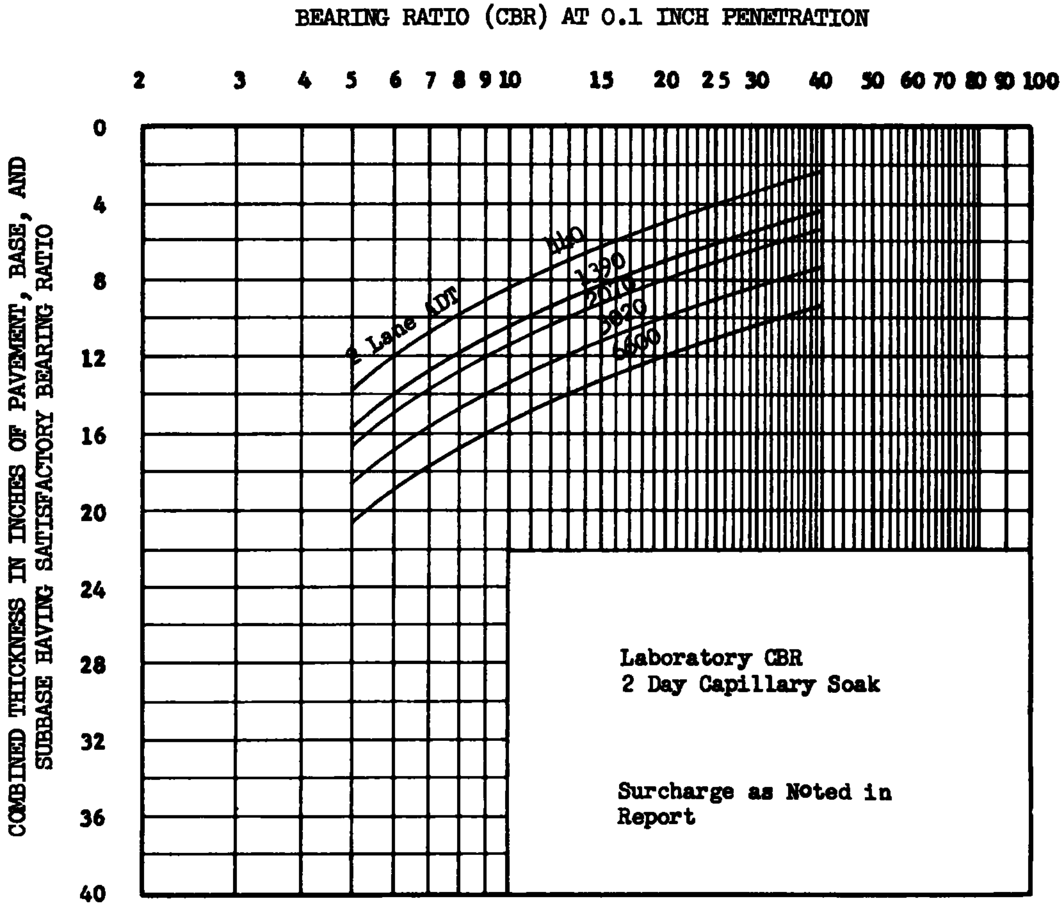


Figure 8. Design curves developed from service rating zones for a condition index of 60.

Figure 9 shows the design curve for the three EWL ranges and the CBR test conditions.

The design curves (as noted previously) resulted from the study of three EWL ranges with a limited amount of data. It follows that the curves are, at best, good estimates of the thickness requirements. The curve for an EWL range of 5 to 15 x 10^6 was developed from eight test sites where three of the eight sites failed. The design curve based on the EWL range of 19 to 33 x 10^6 was developed from thirteen test sites. Although a scattering of points resulted, the design curves should be fairly realistic. The design curves resulting from a study of the EWL range of 36 to 60 x 10^6 were developed from six failed sections. Inasmuch as a number of the sections had condition index numbers of 60 to 65 the design curves were placed immediately beneath the lowest points.

Because complete traffic data were available for the test sites, it was possible to calculate the average two-lane ADT from the EWL groups. The volumes are given in Table 5.

Comparison of Design Curves

Figure 10 shows the design curves which resulted when the service-rating curves were adjusted to volumes corresponding to the actual average volumes obtained from the equivalent wheel-load study (Table 6). The three adjusted curves were superimposed on the results of the EWL study.

TABLE 4

EQUIVALENT WHEEL LOADS AND RELATIVE DATA

Traffic Class	Test Site Number	Total Equivalent 5,000-lb Wheel Loads 1948 - 1958	Average 2 Lane Volume—ADT	Condition Index	Service Rating T_S=1,850
1	51	904,006	125	75	47
2	55	4,357,709	396	79	65
2	84	7,210,139	400	56	35
3	70	7,493,889	725	76	67
3	71	7,493,889	725	68	57
4	86	13,359,667	800	68	58
4	82	6,004,561	824	82	75
4	83	15,566,607	870	38	28
4	78	13,847,791	1,190	48	42
5	68	19,343,560	1,535	68	66
5	87	19,343,560	1,535	71	69
5	72	27,878,509	1,892	76	76
5	73	27,878,509	1,892	70	70
5	65	21,234,307	1,948	59	60
5	66	24,329,003	2,088	48	50
6	67	28,491,134	2,378	40	43
6	62	33,173,391	2,685	41	46
6	63	33,173,391	2,685	71	75
6	81	30,667,717	2,743	70	75
6	85	29,385,712	2,743	79	83
6	56	26,542,419	2,812	64	69
7	58	31,378,346	3,220	60	67
7	64	41,365,061	3,221	60	67
7	54	42,845,530	3,248	65	72
7	79	38,950,933	3,420	57	65
7	80	41,094,096	3,616	66	74
7	57	36,670,335	3,648	54	63
7	77	60,644,152	4,568	57	69

The design curves are for the laboratory CBR of the two-day capillary soak specimens. Although the curves are not identical, close agreement did exist. It can also be seen that the shape of the original CBR design curve was generally duplicated.

The design curves (Fig. 11) have been adjusted to take into consideration the results of both the service rating and EWL studies. The curves have been extended to cover the range of CBR values normally encountered in Florida.

Current Studies

Additional work is currently being done on the development of a semi-empirical method of design. This method will consider in addition to the CBR, tests with larger plates, subgrade modulus, and the Burmister layered theory. Tests are in progress to determine seasonal variation in moisture contents of the pavement section, plate bearing tests of the layers and deflections in the pavements under load as measured with the Benkelman beam.

PROPOSED DESIGN CURVES AND SECTION DESIGN

The flexible pavement design curves proposed for the State of Florida are shown in Figure 11. The curves are a result of the service rating and equivalent 5,000-lb wheel-load design studies completed to date. The design curves are related to volume

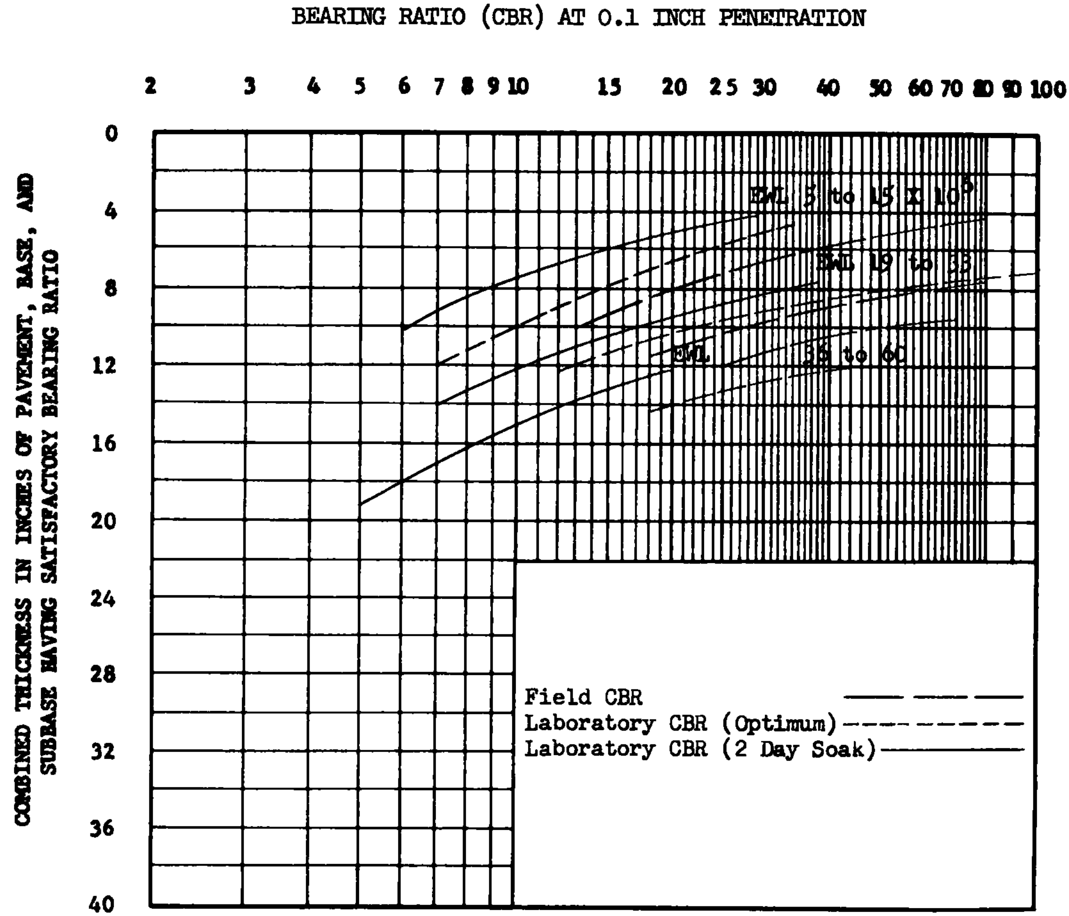


Figure 9. CBR design curves developed from EWL groups.

(10-yr, 2-lane ADT) and equivalent wheel load (10-yr, 5,000-lb wheel loads for 2 lanes). Average daily traffic (ADT) and EWL data were determined by the Traffic and Planning Division (10) for the flexible pavement design study and can be estimated by that division for any proposed highway. The design curves include four major traffic groups. Three of these groups are the direct result of the analysis of performance data, the fourth curve has been extrapolated from the resulting original curves. The design curves are for the estimated 10-yr traffic conditions given in Table 6.

TABLE 5

AVERAGE ADT, 2 LANE VOLUME, FOR EACH EWL RANGE

EWL Range	Average ADT
5 to 15 x 10^6	742
19 to 33 x 10^6	2,321
36 to 60 x 10^6	3,622

The proper design curve for any given traffic condition, considering both volume and EWL would be the curve indicating the maximum thickness. As an example, if the estimated volume were 2,000 v.p.d. and the estimated EWL was 38 x 10^6, this would result in the use of design curve C for heavy traffic even though the volume indicated medium traffic. After selection of the proper design curve, the design of the section would follow normal flexible-pavement design principles.

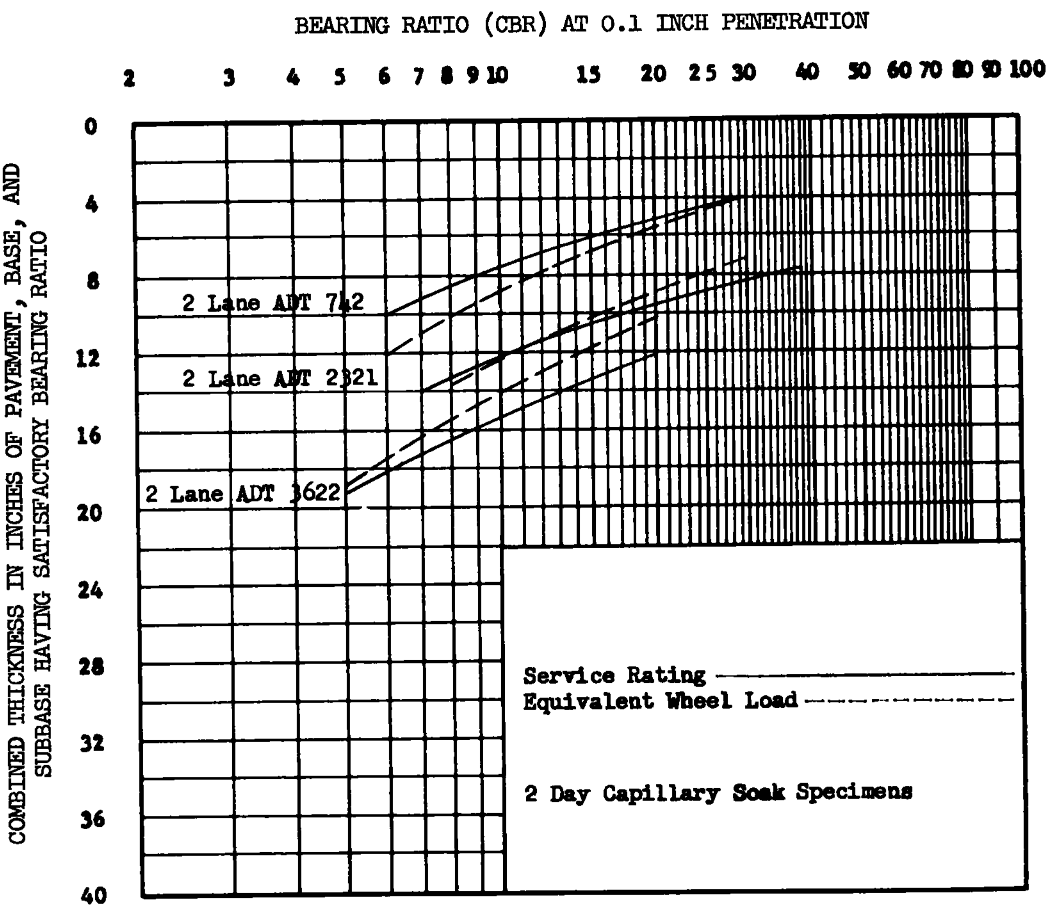


Figure 10. Comparison of design curves developed from service rating and EWL.

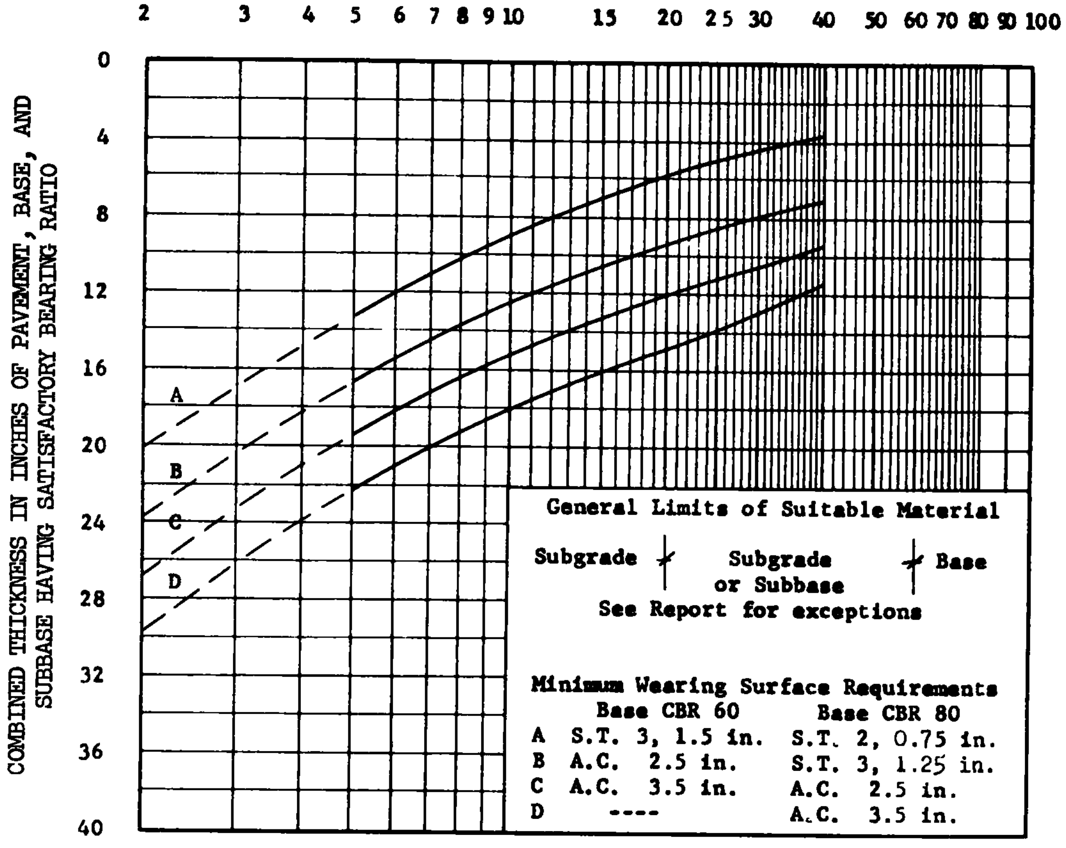
Surface Requirements

The wearing surface requirements are noted again in Table 7. The requirements are discussed briefly in this report and no further comments are added.

Base-Course Material Requirements

The requirements of the base material are given in Table 8. These are the minimum CBR requirements based on the flexible pavement study and base studies performed by Florida. Other state requirements such as chemical analysis, gradation, plasticity, and compaction should be adhered to.

The aforementioned requirements are for all materials used as base-course materials. The major flexible-pavement design study reported herein dealt with limerock base-course pavements; however, preliminary work was done with sand clay and other commonly used base-course materials (6). The CBR requirements were found to be generally the same. The use of a sand-clay base-course material having a two-day soak of 40 for light traffic should be avoided. Sand clay will continue to gain in moisture content beyond the two-day period with a resulting significant loss in bearing. Sand clay base materials should have a minimum bearing value of 60 for light, medium, and heavy traffic.



CLASSIFICATION OF TRAFFIC

CLASS	2 LANE ADT Estimated 10 Year	EWL GROUP(5000#) Estimated 10 Year Total
A. LIGHT	500 to 1500	5 to 15 X 10^6
B. MEDIUM	1500 to 3000	16 to 35 X 10^6
C. HEAVY	3000 to 4000	36 to 50 X 10^6
D. VERY HEAVY	4000 to 6000	51 to 80 X 10^6

Figure 11. Flexible pavement design curves—CBR in percent at 0.1-in. penetration, 2-day capillary soak specimens; compacted at optimum moisture and maximum density (modified AASHO surcharge as noted in report).

Subbase Requirements

The minimum required subbase CBR should be at least 20. Since additional thickness of surface and acceptable base course material can be substituted for inadequate bearing value subbase material, the lower limit cannot be established accurately; however, the lower limit of 20 should be used. Desirable subbase material should have a CBR of about 30. All subbase material should be compacted at optimum moisture and maximum density and tested after a two-day capillary soak period. A surcharge of 15 lb should be used. All other state specifications pertaining to grain size and plasticity requirements should be adhered to.

TABLE 6

DESIGN CURVE AND TRAFFIC DATA

Design Curves	Class of Traffic	Volume[1] 2 Lane ADT (11-yr tot.)	EWL 5,000 lb[1] 2 Lane (11-yr tot.)	Design Volume 2 Lane ADT (10-yr tot.)	Design EWL 5,000 lb 2 Lane (10-yr tot.)
A	Light	742	5 to 15 x 10^6	500 to 1,500 750	5 to 15 x 10^6
B	Medium	2,321	19 to 33 x 10^6	1,500 to 3,000 2,000	16 to 35 x 10^6
C	Heavy	3,622	36 to 60 x 10^6	3,000 to 4,000 3,500	36 to 50 x 10^6
D	Very heavy			4,000 to 6,000 5,000	51 to 80 x 10^6

[1] Actual data from flexible pavement design study.

TABLE 7

MINIMUM RECOMMENDED WEARING SURFACE TYPES AND APPROXIMATE THICKNESS

Class of Traffic	Type of Wearing Surface	Thickness (in.)
Light	Double surface treatment (S.T. 2)	0.75
Medium	Triple surface treatment (S.T. 3)	1.25
Heavy	Asphaltic concrete (binder + surface)	2.5 to 3.5
Very heavy	Asphaltic concrete (binder + surface)	3.5 to 4.0

TABLE 8

BASE MATERIAL REQUIREMENT (CBR)

Class of Traffic	CBR of Base Material	Thickness
Light	(40)[1] 60	As shown on design curves
Medium	(60)[2] 80	
Heavy	(60)[2] 80	
Very heavy	80	

[1] When base material with a CBR of 40 must be used, additional thickness of wearing surface should be provided. A CBR of 40 is permissible only for light traffic.

[2] Where base material with a CBR of 60 must be used, additional thickness of wearing surface should be provided.

Note: CBR determined by testing a sample compacted at optimum moisture and maximum density (Modified AASHO) and tested without surcharge after two days of capillary soaking.

Subgrade Requirements

No minimum subgrade CBR is required. The laboratory CBR should be established by generally duplicating field compaction of the subgrade. The maximum subgrade CBR should be established by compacting specimens at optimum moisture and maximum density and testing after a two-day capillary soak period. A surcharge of 20 lb should be used. All other state requirements pertaining to suitability and compaction should be adhered to.

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