Application of AASHO Road Test Results to Design of Flexible Pavements in Minnesota

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This paper reports on a study of the design of flexible pavements in Minnesota applying some of the concepts of the AASHO Road Test. The method presently used to design flexible pavements in Minnesota is described; it utilizes ADT and HCADT, a designation of spring axle load, the AASHO soil classification system, and gravel equivalent factors. The design method was studied by measurements on 50 test sections established on in-service highways throughout the state. The program of field plate and Benkelman beam tests and laboratory strength tests on the pavement materials is described. An important aspect was variation in strength throughout the year.

The performance of the sections has been studied by utilizing the roughometer in determining PSI; traffic has been evaluated; performance trends are noted, and predictions have been made. The AASHO Interim Guide and Asphalt Institute design procedures, which are based on the concepts of the AASHO Road Test, are used as a basis of comparison for the performance of the Minnesota test sections. A relationship using the spring Benkelman beam deflection and performance, originally developed at the Road Test, is examined and the Minnesota results thus far are studied for compliance. Modifications of the present Minnesota flexible pavement design are suggested on the basis of the observed performance of the test sections to date.

•FOR THE PAST several years the Minnesota Department of Highways has engaged in a study of its flexible pavement design utilizing some of the concepts developed at the AASHO Road Test. The project is being conducted under the Highway Planning and Research Program financed jointly with federal-aid funds of the U. S. Department of Transportation, Bureau of Public Roads, and with state funds of the Minnesota Department of Highways. A part of this work has been done by personnel of the Civil Engineering Department of the University of Minnesota under a contract with the Department of Highways. This paper is a report based on the work of both the Department of Highways and the University.

The present Minnesota design procedure considers traffic in terms of average daily traffic (ADT) and heavy commercial average daily traffic (HCADT), the AASHO soil classification groups (A-1, A-2, etc.), and gravel equivalencies for various pavement materials. The AASHO Road Test has given relationships that make it possible to predict the number of applications of a given axle load that a given pavement can withstand before being reduced to various levels of serviceability. To use these relationships in a study of the Minnesota design method, it was necessary to collect information on

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Minnesota pavements that could be entered into these equations. It was decided that in-service highways would be used in such a study for the following reasons:

1. It is possible to obtain performance information more quickly by using in-service highways;
2. The state can be covered more thoroughly by using individual sections in different areas;
3. A variety of embankment materials can be studied more easily;
4. The effect of various levels of traffic density can be studied as they occur in service; and
5. Using in-service highways is less costly than building a separate test road.

Fifty in-service sections on Minnesota highways were selected for study in 1963 and 1964. Most of these are 1200 ft long and one traffic lane wide. Serviceability tests are made on two 500-ft long sections and a 200-ft section is used for sampling or destructive tests. Plate tests, for example, are run in the 200-ft section. The test sections are located throughout the state and represent a range of soils, climate, and traffic. All are of essentially modern design. The surface thicknesses vary from 1.5 to 9.5 in., the granular bases from 0 to 15 in., and the granular subbases from 0 to 29 in.

The work that has been done on these sections is as follows: (a) the subgrade soils, subbase, base, and surface materials have been sampled and tested in the laboratory both by routine tests and also with special strength tests; (b) field strength tests such as plate bearings and Benkelman beam tests have been made periodically; (c) serviceability determinations have been made yearly; and (d) traffic information has been collected.

Along with the general concept of correlating the performance of Minnesota asphalt pavements to the AASHO Road Test, the establishment of the test sections has made it possible to make various other analyses that when used with Minnesota Department of Highways practice will have immediate use. An example of these programs is the spring recovery program, for which strength tests have been run every week or so on 15 to 20 test sections during the critical spring period. The Benkelman beam deflection test has been established as a useful tool for determining the maximum allowable spring axle load. The use of the R-value for evaluation of soils in the present Minnesota Department of Highways design procedure has also been recommended based on the results of the embankment testing. Methods of using traffic and the serviceability concept (in the present design procedure) have been conceived. A roughometer calibration wheel has also been set up in conjunction with this study. Thus, there already are benefits derived from the investigation.

Using the serviceability determinations and the traffic information, the performance trends of the sections are being studied. Because the history is only a few years old, however, a final evaluation of performance of the sections cannot be made. The performance to date is being studied on the basis of the design procedures in the AASHO Interim Guide and the Asphalt Institute method.

PRESENT MINNESOTA FLEXIBLE PAVEMENT DESIGN PROCEDURE

Since 1954, flexible pavements in Minnesota have been designed essentially using the traffic and the type of embankment soil to determine a design thickness. Also specified for each design is the maximum allowable spring axle load in tons. The thickness specified is in terms of a gravel equivalent that rates the various components of the pavement section.

The traffic loading is considered using allowable spring axle load categories for light traffic, and the ADT and HCADT. The ADT includes all vehicles and the HCADT includes all trucks with six or more tires; thus HCADT does not include small pickup and panel-type trucks. The design makes provision in two cases where the allowable spring axle load can be increased by 2 tons if an additional 2- or 3-in. overlay is added at a later time; thus a 7-ton design can ultimately become a 9-ton design (Table 1).

The ADT and HCADT used for design are values predicted for 20 years in the future. To predict the traffic 20 years from now, it is assumed that the volume increases about

TABLE 1

MINNESOTA FLEXIBLE PAVEMENT DESIGN STANDARDS, 1964[a]

Axle Load	Daily Hvy. Com. (HCADT)	Total Daily Veh. (ADT)	Surface		Bituminous Base		Bituminous Treated Base		Gravel Base, Spec. 3138		Sand-Gravel Subbase, Spec. 3123		Total Base Thickness (in. of G. E.)	Total Pavement Thickness (in. of G.E.)
			Thickness (in.)	Spec.	Thickness (in.)	Spec.	Thickness (in.)	Spec.	Thickness (in.)	Class	Thickness (in.)	Class		
5 ton		Less than 400	1½	2321					3	5	5	4	7	9
7 ton		Less than 400	1½	2331					4	5	6	4	8½	11½
5 ton—ult. 7 ton		400 to 1,000	1½	2331	1	2208			3	5	6	4	9	12
7 ton		400 to 1,000	2	2331	1	2208			3	5	8	4	10½	14½
7 ton—ult. 9 ton	Less than 150	Less than 1,000	2	2208					4	5	9	4	11	14
7 ton—ult. 9 ton	150 to 300	1,000 to 2,000	2	2331	1	2208			5	5	9	4	13½	17½
9 ton	Less than 150	Less than 1,000	2	2331	1	2208			5	5	9	4	13½	17½
9 ton	150 to 300	1,000 to 2,000	3	2341	1	2208			5	5B	10	4	14	20
9 ton	300 to 600	2,000 to 5,000	3	2341	1½	2331	4 Rich	2204			6 6	4A 4	18	24
9 ton	600 to 1,100	5,000 to 10,000	3½	2351	3½	2331	4 Lean	2204			6 6	4A 4	21	28
9 ton	More than 1,100	More than 10,000	3½	2351	4½	2331	4 Lean	2204			6 8	4A 4	24½	31½
9 ton	More than 1,100	More than 10,000	3	2351	8 Concrete				3	5	3	4	—	—

[a]These designs are for use on A-6 subgrade soils; for use on other soils, thicknesses should be adjusted as described in Table 2.

TABLE 2

SOIL FACTORS FOR MINNESOTA DEPARTMENT OF HIGHWAYS FLEXIBLE PAVEMENT DESIGN PROCEDURE

AASHO Classification of Soil	Soil Factor, Percent	AASHO Classification of Soil	Soil Factor, Percent
A-1	50 to 75	A-5	130+
A-2	50 to 75	A-6	100
A-3	50	A-7-5	120
A-4	100 to 130	A-7-6	130

3 percent per year. This is equivalent to multiplying the present values by 1.805. Local conditions are also considered and the projected value may either be increased or decreased, based on the potential use of the road.

The design thicknesses listed in Table 1 are for A-6 type embankment soils, which are the predominant soils in most of Minnesota. The designs are modified for other embankment types using the percentages given in Table 2. These percentages reflect both the strength and the frost susceptibility of these soils relative to the A-6 soils. There are ranges of percents shown for A-1, A-2, and A-4 soils. It is therefore possible to use some judgment relative to capabilities of these soils.

The percentages given in Table 2 are applied to the base plus subbase portion of the gravel equivalents for the A-6 soils. The plus or minus variation in gravel equivalent is applied to the subbase thickness only. For a given set of traffic conditions, the surface and base thicknesses for any type of embankment are the same or greater than the thicknesses shown for the A-6 embankments. If a particular design is calculated to require less gravel equivalent than is provided by the base course, the base thickness shown for the A-6 soils for the given level of traffic is still used. In other words, the surface and base thicknesses shown for A-6 soils in Table 1 are minimum designs for the respective values of traffic.

The gravel equivalent factors used for evaluating the various materials and mixtures in a flexible pavement section are based on a value of 1.00 for gravel base (Spec. 3138, Class 5 or 5B). The factors go as high as 2.25 for plant-mix surface (Spec. 2341 or 2351) and as low as 0.75 for sand-gravel subbase.

PROPERTIES OF PAVEMENT SECTION COMPONENTS AND EMBANKMENTS

All materials of the layers making up the pavements on the 50 test sections have been sampled and tested. Extractions have been made of asphalts from cores of the pavement and such tests as penetration, ductility, and softening point are available for further study.

The tests made on the base and subbase materials include field densities and moisture contents, gradations and plasticity indexes. A study was made to compare the strength of some of the granular materials as determined by three different test procedures. The three tests used were the triaxial test, the California Bearing Ratio (CBR) and the Hveem stabilometer test.

Five base and five subbase materials were selected. Four of the materials represent extremes of T 99 density and of the percent passing the No. 200 sieve. The fifth material chosen represented values between these.

The results of the strength tests run on the 10 samples with the triaxial test, the stabilometer R-value, and the CBR were compared, and the following conclusions are drawn. The angle of internal friction does not vary significantly for any of the granular materials that pass the Minnesota Department of Highways specifications. The CBR results order the granular base materials in a reasonable fashion, but to obtain a design CBR value it is necessary to run a number of tests. The stabilometer R-value has also shown consistent results and it is an easier test to run than either the triaxial or CBR tests. It was, therefore, decided to run R-value determinations on all of the granular base and subbase materials. There were only two base materials with an R-value below 76 and only three subbases below 74.

Extensive testing was also done on the embankment soils, including determination of field density and moisture content, gradation, plasticity index, moisture-density tests, CBR, and stabilometer R-value. The soils range from A-1-b to A-7-6 based on the AASHO classification, and texturally from sands and gravels to heavy clays. The group indexes vary from 0 to 20. The AASHO Road Test embankment was an A-6 soil with a group index of about 8.

The stabilometer R-value test was run using the method outlined by Wolfe (2). This method is essentially the same as that recommended by the California Division of Highways and the Asphalt Institute. An exudation pressure of 240 psi is used to establish the design R-value. It was found that this exudation pressure yielded a specimen with a moisture and density close to what was felt were critical field conditions (2). The distribution of R-values shows a concentration of values below 20. The distribution of these strengths for soils throughout the state are most likely of this same order. The values range from a low of 5 to a high of 77, which indicates about a maximum range based on this test. The R-value of the Road Test embankment has been taken as 12 (3). Correlations are made between the R-value at various exudation pressures and field strength of the embankment based on the plate tests run on the embankment. It was found that the R-value determined at 200- to 240-psi exudation pressure correlated best with the field embankment strength measured with the plate load test.

The CBR test was also run on each of the embankment soils (4, chap. 8). This is essentially the Corps of Engineers CBR test method. Compactive efforts of 26 blows per layer and 55 blows per layer were used for establishing the CBR curves. The design CBR was taken to be the lowest CBR value obtained assuming the embankment was compacted according to Minnesota compaction specifications. The Minnesota specifications state that the top 3 ft of the embankment is to be compacted to maximum T 99 dry density and between 65 and 102 percent of optimum moisture content. The design CBR's range from 1 to 65. The CBR of the AASHO Road Test embankment has been estimated at 2.5 to 4.0, depending on the analysis of the lab data.

FIELD STRENGTH TESTING

The strength of the Minnesota test sections has been evaluated in the field using two strength tests—the plate bearing test and the Benkelman beam deflection test. The plate bearing test has been run using two procedures, the standard Minnesota "quickie" procedure and a repeated load procedure. The quickie method consists of applying loads in several increments to the pavement on a circular 12-in. steel plate and recording deflections. This test has been run on most of the test sections once a year and more often during the critical spring period on test sections selected for spring recovery study. Overall, there has not been a significant change in bearing value from year to year except for the summer of 1965, which had lower values, following the most critical spring thus far observed.

A series of plate load tests, termed "fractional repeated load tests", was run once on every section in 1963 or 1964. In this procedure, repeated load plate tests were run on the surface, the top of the base, the subbase, and the embankment. The test procedure is the same as that used at the AASHO Road Test (1).

The data obtained from a repeated load test consist of total and elastic (rebound) deflections for each application of each load. The first value of total deflection is usually larger than the other two, probably because of seating effects. The rebound deflection does not usually vary by more than 0.01 or 0.02 in. for the three applications of load. E-moduli of the embankments have been calculated using the average elastic deflection for each magnitude of load. The E-modulus is calculated from Boussinesq's equation for a single elastic layer. The E-moduli of the granular embankments tend to increase with increasing pressure, and the E-moduli of the plastic embankments tend to decrease with increasing pressure on the plate.

Repeated load plate tests have been run periodically on the surface of each of the test sections. On all but the spring recovery test sections the tests have been run annually. On the spring recovery sections, repeated load plate tests were run at the same time as the quickie plate tests, which was about every week during the critical spring period.

Benkelman beam deflection tests were run on all test sections in two sets in 1963 and 1964. The summer tests were run in June through August, and the fall tests were run in September and October. In the following years deflection tests were run about once a year in most of the sections. To better define the strength during the critical spring period, deflection tests were run on the spring recovery sections about weekly.

Two procedures for measuring deflections from a 9-ton axle load were used in 1963. The "normal" and "rebound" methods are described in detail elsewhere (1). Also, in 1963, deflections were run in both wheelpaths. In 1964 and following years only the rebound procedure has been used and only in the outer wheelpath. The rebound procedure was used because the effect of the deflection basin on the readings has been found to be less by the Canadian Good Roads Association (6). The outer wheelpath has been used because comparison between the outer and inner wheelpaths showed the outer wheelpath to have higher deflections.

A deflection for a test segment is determined by running deflection tests at 50-ft intervals in each 500-ft segment. The individual values are averaged and a standard deviation is determined.

VARIATION IN PAVEMENT STRENGTH THROUGHOUT THE YEAR

In addition to the annual strength testing on the Minnesota test sections, a number of sections were tested at more frequent intervals and especially during the critical spring period. Pavement strengths determined with the quickie plate load test have been used as criteria for establishing maximum allowable spring axle loads for some 10 to 15 years. A study was run in 1956 and 1957 to establish the variation in pavement strength during the year according to the quickie plate test. The variation in pavement strength was found to be dependent on embankment type and not dependent on pavement section thickness (5).

The more frequent testing on the test sections has been used to check the relationships previously used with the plate test, and to establish spring ratios of Benkelman beam deflections so that the strength measured at any time of the unfrozen portion of the year could be used to predict a spring strength.

The test sections selected have predominantly clayey subgrade soils, which are of greatest interest because this type of soil is most prevalent in Minnesota. Another criterion used in determining the test sections was that the testing crew be able to cover the entire testing circuit in about one week. During 1964 and 1965, 11 test sections were selected to be studied, and in 1966, 1967, and 1968, 15 test sections were included in the program. The spring recovery field testing included the bearing value using the Minnesota quickie plate load test; the Benkelman beam deflection test; moisture contents of the base, subbase, and embankment soil; and air and mat temperatures.

The quickie plate load test was not run in 1968 because it was felt that more information could be obtained faster with the Benkelman beam deflection test. The procedures mentioned previously have been used for the plate load and the Benkelman beam tests. The data studied included climatic variables of temperature and precipitation. These data were considered to give some idea of how critical the years of spring recovery testing were relative to an overall average year. If testing is continued for a number of years on the same sections, it will be possible to estimate the frequency of various critical years in terms of loss of strength.

Both the quickie plate load test and the Benkelman beam deflection test have been used to evaluate bearing capacity of flexible pavements. However, the two methods apparently measure different characteristics of strength. The Benkelman beam test results are more dependent on the upper portion of the pavement, whereas the quickie plate load test results are more dependent on the lower structure. This is reasonable because the quickie plate load test evaluates bearing capacity at a higher deflection than does the Benkelman beam test. It is concluded from the 4 years of spring testing that the Benkelman beam test is easier to run and that more tests can be made in a given length of time. It is felt that the beam devices could be efficiently used in each Department of Highways district to evaluate the roads in the districts, whereas the expenditure involved with providing a plate bearing truck for each district would be prohibitive.

TABLE 3

DEFLECTION RATIOS TO CALCULATE MAXIMUM SPRING DEFLECTIONS FROM DEFLECTIONS TAKEN DURING OTHER NONFROZEN TIMES OF THE YEAR

Asphalt Surface Thickness	Date of Test								
	Sept.	Aug. 16-31	Aug. 1-15	July 16-31	July 1-15	June 16-30	June 1-15	May 16-31	May 1-15
2 in. or less	1.85	1.80	1.75	1.70	1.65	1.60	1.50	1.35	1.15
2½ to 3½ in.	1.80	1.78	1.75	1.70	1.65	1.60	1.50	1.35	1.15
3½ to 5½ in.	1.75	1.70	1.65	1.60	1.55	1.50	1.45	1.30	1.15
5½ to 8 in.	1.45	1.42	1.40	1.37	1.35	1.32	1.30	1.20	1.10
Greater than 8 in.	1.25	1.20	1.15	1.10	1.08	1.05	1.05	1.05	1.00

Ratios given are for clayey embankments. For loam and silt loam embankments, 0.15 higher ratios are recommended from June 15 through September and 0.10 higher from May 1 through June 15. For sand or sand and gravel embankments, a ratio of 1.20 is recommended from June 1 through September, 1.10 from May 15 to June 1, and 1.05 prior to May 15.

With regard to the Benkelman beam deflection test, the following conclusions were drawn:

1. The percent of maximum spring to fall deflection values generally decreases as the surface mat thickness increases.
2. The loss of strength is dependent on the embankment type. A division into three soil-type categories of plastic, semiplastic, and nonplastic is appropriate.
3. One test section with a crushed limestone base material showed high loss of strength. This is evidently because the material had a high percentage of fines.
4. The average percent of maximum spring to fall deflection values compared closely to the value obtained from the CGRA study (6). However, that study does not show deflections to be dependent on surface thickness.
5. The highest average annual Benkelman beam deflection occurred during the spring that showed the highest average moisture content in the subbase and embankment.

The average date of minimum strength occurred 18 days after the end of the freezing season. The longest time period between the end of the freezing season and minimum strength occurred during the year that had the highest freezing index value. These results occurred for both test methods.

Based on the spring recovery study and other information available from previous studies by the Minnesota Department of Highways, three methods have been suggested for the determination of allowable spring axle loads.

The first method uses quickie plate bearing test results and is basically the standard method that had been used by the Department of Highways. Less reduction in spring bearing percentages is recommended for thicker pavements.

The other two methods use the Benkelman beam deflection test to evaluate the strength of the pavement. The first deflection method uses the deflection results to correlate with the plate bearing value and then this value is converted to an allowable spring axle load as presently used. This method is of doubtful value because the correlations between deflection and bearing value are only approximate. The third method uses the Benkelman beam deflection and allowable deflections to establish the allowable spring axle load without a correlation with the plate bearing value. The method requires the determination of three factors to estimate an allowable axle load: (a) the variation of deflections throughout the year for various highway sections; (b) the determination of allowable deflections; and (c) the relationship between load and deflection for appropriate highway sections.

The variation in deflection throughout the year has been determined from the spring recovery study. A deflection measured at any time during the year can be converted to a spring value using the ratios in Table 3. These ratios were developed by plotting deflections for each test section for each year (1). The spring of 1965 was found to be the most critical spring and thus the ratios shown in Table 3 are close to the ratios found for that year. By grouping the sections into various categories of thickness, it was found that the ratios were most dependent on surface thickness. This is reasonable because in the spring the surface mix tends to be harder when cool and thus would tend to decrease the spring deflection. The ratios were also assumed to be dependent on

TABLE 4

SUMMARY OF RECOMMENDED ALLOWABLE SPRING DEFLECTIONS

One-Way Daily N18[a]	Two-Way ADT	Two-Way HCADT	Allowable Deflection (in.) Where Surface Thickness Is:		
			Less Than 3 in.	3-6 in.	Greater Than 6 in.
25	500	50	0.075	0.065	0.055
25-50	500-1,000	50-100	0.070	0.060	0.050
50-150	1,000-3,000	100-150	0.060	0.050	0.040
150	3,000	150	0.045	0.040	0.035

[a]N18 = Daily equivalent 18,000-lb axle loads.

embankment soil classification. The spring recovery section that had a sand embankment (TS 15) had a very low deflection ratio, which agrees with the previous findings using the plate load test (5). Test Section 41, which has a loam embankment soil, showed very erratic behavior; this had also occurred for this type of soil in previous strength studies. Thus the ratios for this group (semiplastic soils) are the highest.

A number of previous studies (7, 8, 9, 10) along with the AASHO Road Test have been considered to establish allowable spring deflections. The measured maximum spring deflections from the spring recovery study were considered relative to the performance of the sections to help establish allowable deflections. The allowable deflections that result from these considerations are shown in Table 4. The allowable deflections have been assumed to be dependent on the surface thickness and the level of traffic on the road, as indicated in the table. The traffic values in Table 4 are roughly correlated using the relationships developed in the traffic study related to this project and reported elsewhere in this Record (16).

Using the data reported by Huculak (10) and some data from the Minnesota sections plus some load-deflection relationships from the AASHO Road Test, it has been assumed that deflection and load are directly proportional for a relatively well-designed road and loads within the legal axle limit.

With the assumption of a linear relationship between load and deflection used, three quantities are needed to calculate the allowable spring axle load. These are (a) the axle load under which the deflection test is made, (b) the predicted spring deflection, and (c) the allowable spring deflection for that section of road. The allowable spring axle load can then be calculated by

$$L_A = L_D \frac{d_A}{d_s} \tag{1}$$

where

L_A = allowable spring axle load, tons;
L_D = axle load used for deflection testing, tons;
d_A = allowable maximum spring deflection from Table 4, in.; and
d_s = predicted spring deflection for the pavement section, in.

It has been recommended that the spring recovery work be continued so that the relationships thus far developed can be verified.

EVALUATION OF PRESENT CONDITION OF PAVEMENT SECTIONS

The terms of present serviceability rating (PSR) and present serviceability index (PSI) were developed at the AASHO Road Test (15). Equation 2 is the relationship found at the Road Test for flexible pavements:

$$PSI = 5.03 - 1.91 \log (1 + SV) - 1.38 \overline{RD}^2 - 0.01 (C + P)^{1/2} \tag{2}$$

where

PSI = present serviceability index;
SV = slope variance;
RD = rut depth, in.; and
C + P = area of cracking and patching, sq ft per 1,000 sq ft.

These measurements were made every 2 weeks at the Road Test and the rate of decrease of PSI with traffic was used to define performance. Formulas reported from the Road Test define the thickness required to maintain the PSI above a given level for a given number of applications of a particular axle load. The terminal level of PSI for design purposes is usually taken as 2.50 or 1.50. A value of 1.50 was considered failure at the Road Test.

To use the concepts developed at the Road Test it is desirable to establish a serviceability determination for highway pavement sections and study the decrease in value of this serviceability with time. This concept can also be a useful tool for routine evaluation of the condition of highway pavements.

To apply the results of the AASHO Road Test to other areas it is necessary to use the same parameters as were developed in the Road Test. This required, in the case of determination of road condition, that a measure of present serviceability for pavements be devised. A direct correlation is difficult in Minnesota for two reasons. First, a profilometer to measure slope variance was not available. Second, the surface textures on in-service highways differ from those at the AASHO Road Test. A third more basic consideration is that some engineers feel that the present serviceability index, as developed at the Road Test, does not always directly reflect the strength of a pavement structure. An example of this is a case where roughness occurs because transverse cracks have developed and become rough.

A BPR-type roughometer has been used to evaluate roads in Minnesota for about 20 years. Because this device was operational and a reasonably good correlation was found between the roughometer and serviceability at the AASHO Road Test, it was decided to relate roughness measurements to serviceability ratings using the Minnesota Department of Highways roughometer. The correlation has been obtained by special runs made in South Dakota in 1960 through 1962 (11), by a special test between the CHLOE profilometer and the roughometer in Minnesota in 1961 (14), in runs made on sections in Indiana established by Purdue University (12), and in a special study of about 200 half-mile segments selected in Minnesota. The details of all these correlation studies cannot be given here. However, the final equation evolved for determining the present serviceability index is as follows:

$$PSI = 11.03 - 3.98 \log (RI) - 1.38 \overline{RD}^2 - 0.01 (C + P)^{1/2} \qquad (3)$$

RI is the roughometer index in inches per mile and the other items are as noted before. This equation is similar to one developed by the Illinois Division of Highways (13) for their roadometer. To calculate the PSI of the test sections using Eq. 3 it is necessary to determine the roughometer index, the rut depth, and the cracking and patching of the pavement section.

The roughometer electronics may vary somewhat from time to time. A calibration course along TH 96 near Stillwater had been used for calibration of the roughometer. Because this is a typical highway pavement, the surface tends to vary in roughness with time. To establish a more consistent calibration for the roughometer, a 10-ft diameter vertical steel wheel located at the Rosemount Research Center was reactivated during the fall of 1964. A 15-hp AC motor was connected to the existing vari-drive transmission and a set of pulleys so that the wheel runs at 57 rpm, which is equivalent to 20 mph at the circumference. A system has been arranged so that the roughometer truck can be backed up to the wheel and the roughometer wheel placed on the calibration wheel. The calibration wheel is fitted with nine ¼-in. strips on one side, and three ¼-in. strips on the other side. When the wheel of the roughometer is set on the nine-strip side, a roughness of approximately 100 in. per mile results; the three-strip side gives about

60 in. per mile. If the calibration varies by more than 10 percent at any time, the roughness indexes measured in the field must be adjusted.

The other two elements of the serviceability index (rut depth and cracking and patching) have also been determined periodically on the Minnesota test sections. Extreme precision is not necessary because the rut depths usually encountered do not affect the PSI significantly. Cracking and patching values are estimated in square feet per 1000 square feet of pavement area. The structural cracking, or that caused by repeated wheel loads, is the only type considered for serviceability calculations. Shrinkage and contraction cracks that are not load-associated are not included in the area of cracking. Load-associated cracking is divided into three types (15): Class 1 cracking is that which has connected into blocks smaller than about 2 ft in dimension; Class 2 cracking is that which has connected into blocks less than 6 in. in size; and Class 3 cracking is Class 2 cracking with pieces that are loose. Class 2 cracking is also called alligator cracking. Classes 2 and 3 cracking only are included in the C + P term of the PSI formula. This term has only a small effect on the value of the PSI and therefore need be only approximated.

Because the general decreasing trend of the serviceability of the Minnesota test sections is slight, it has only been necessary to determine serviceability once per year. Present serviceability indexes have been determined on each test section in each of the summers of 1963 through 1967 using the roughometer and determinations of rut depth and cracking and patching. The individual values are available for each test segment of each test section and are being used along with the traffic to develop performance trends.

The trends in PSI values for the test segments have been slightly down for the sections that have not been overlaid. The average PSI values for 63 test segments that can be compared for the 5 years are as follows:

Year	Average PSI
1963	3.82
1964	3.76
1965	3.83
1966	3.64
1967	3.63

As can be seen, the general level of PSI has been high and the trend is only very slightly down, which makes a performance analysis based on these concepts somewhat difficult at this time.

Initial PSI values are also necessary for the performance analyses, and they have been obtained from roughometer indexes made when the pavement was constructed. If a roughometer index was not available, an average initial value of 4.10 was used.

DETERMINATION OF A TRAFFIC PARAMETER

To make a performance analysis of flexible pavement sections based on the concepts of the AASHO Road Test, it is desirable to convert the mixed traffic loading on the test sections into a summation of equivalent 18,000-lb single axle loads since the pavement was built. It is also necessary to study the variation in these values on a yearly basis so that equivalent 18,000-lb axle loads can be predicted for the design period of a pavement. A traffic study has been made on the Minnesota test sections to make these determinations possible.

The three purposes of the traffic study can be summarized as follows: (a) to determine the total traffic in terms of the summation of equivalent 18,000-lb axle loads on each of the 50 Minnesota test sections since each was built; (b) to estimate a reasonable growth factor based on the data of the last 12 to 15 years; and (c) to correlate various traffic parameters to equivalent 18,000-lb axle loads for design purposes.

A detailed field traffic study that included weight and volume distributions of vehicles at each test section was made three times throughout 1964 and once during the spring load restriction period in 1965. The data from this study were modified for other years

using statewide data to determine the total equivalent 18,000-lb axle loads on each test section since it was built. The computations were put into a computer program on the CDC 6600 computer.

The rate of increase in equivalent loads has been studied over the last 12 years and it is concluded that a rate of increase of 8 percent per year is reasonable for an overall estimate, but that local conditions can cause a very significant range in growth factor.

The 1964 traffic study was used to correlate ADT, HCADT, and the daily sum of Types 4 and 5 (four-axle and five-axle) trucks to daily equivalent axle loads. The errors in equivalent loads from predicting traffic by these three parameters have been related to gravel equivalent thicknesses. It was found that for design purposes the summation of Type 4 and 5 trucks could best predict equivalent axle loads, but that only a slightly poorer correlation resulted using the HCADT. The error in terms of gravel equivalent is about 1.2 in., which represents about 0.6 in. of asphalt surface.

Using the results of the traffic correlations back to 1956, the correlation between HCADT and daily equivalent loads has been studied and it has been found that the equivalent loads predicted from HCADT over the last 10 years have had an annual increase between 4 and 26 percent, depending on the span of years considered for the increase. Over a 10-year or longer period, an increase of 5 to 8 percent is shown to be appropriate. This increase, along with an annual 3 percent increase in HCADT, yields the growth factor of 8 to 11 percent.

From the results of this study the summation of equivalent 18,000-lb axle loads over a 20-year period represented by the present Minnesota traffic categories shown in Table 1 are calculated (16).

PERFORMANCE TRENDS AND PREDICTIONS

General performance analyses based on the concepts of the AASHO Road Test are made essentially by considering the trend of PSI with the increasing values of summation in equivalent 18,000-lb axle loads. By observing these trends on the test sections it is anticipated that it will be possible to determine how much traffic is required to lower the PSI to a given level (usually 2.5 for primary roads), and to define the shape of the performance curves to aid in the prediction of the number of applications to various levels of serviceability. This can be done by observing the performance trends of the test sections and relating them to the thicknesses and embankment strengths of the sections or to the strength of the sections. At this time the performance of the sections has been compared to the performance predicted by the AASHO Interim Guide and the Asphalt Institute design procedures, which are based on the Road Test concepts. At the Road Test an equation was developed relating spring beam deflections to the number of applications to a PSI of 2.5. Using the measured or predicted spring deflections on the test sections, this relationship is verified for Minnesota conditions. So far there is generally not enough drop in PSI for the test sections to determine thickness or strength requirements absolutely. The relationships shown in this section are the methods that appear to predict performance relationships best from the data available to the present time.

Irick shows a simplified method of predicted log (ΣN18) to a PSI of any level when the original PSI is known and any number of points along the plot are determined (17). A curvature term, B, is calculated and used to make the predictions if more than one point along the line is known. A B-value of 0.5 indicates a curve that is concave down and a B-value of 1.0 indicates a curve that is concave upward.

The equations for this method have been put into a computer program, called PERPRED, that determines the curvature and predicted log (ΣN18) to PSI levels of 2.5 and 1.5. The traffic for the summers of 1963, 1964, 1965, 1966, and 1967 have been used along with the PSI values for those years to enter into the PERPRED program. Most of the test sections had not decreased in PSI enough to yield an accurate estimate of either B or log (ΣN18) to a PSI of 2.5. The B-values generally tend to get higher as the PSI levels decrease. If the initial PSI is high, the B-value tends to be high, indicating upward curvature. At this time it is not possible to establish a well-defined curvature value for the performance trends, but generally it appears that a value close

to 1.0 is appropriate for sections that have lost more than one unit of PSI. For initial estimates of future performance this value can be used.

At the Texas Transportation Institute a method developed at the Asphalt Institute (18) is used to predict performance. For this method the slope of a log (PSI) vs arithmetic $\Sigma N18$ plot is determined and extrapolated to a PSI of 2.5 to predict performance.

Using the same data as used for PERPRED, the last points on the plot are used along with the original point to establish the slope. Again, because of the generally high PSI levels of the sections, the variation in predictions is quite great. Of the 79 test segments for which comparisons were made between PERPRED and this method, only about 21 percent of the predictions were within 50 percent of each other.

Both methods will continue to be used for performance predictions because (a) it is not yet possible to determine which of the models defines the observed performance trends best, and (b) these methods make it possible to predict the applications to a PSI of 2.5 with some degree of accuracy a number of years before the section actually goes to a PSI of 2.5.

The AASHO Interim Guide and Asphalt Institute design procedures are both set up based on the Road Test equations. To check the performance of the Minnesota test sections relative to these, design predictions of life in years left for each test section were made using the traffic level at the time of construction, the thickness of the pavement layers, and the strength of the embankment using the R-value or CBR test value. The number of years of life predicted by these procedures gives an idea of whether a given section is overdesigned or underdesigned according to the procedures.

The following conclusions were made based on a study of these predicted lives:

1. It is not possible to evaluate one of these design procedures against the others because there are not enough test sections that have gone to a PSI of 2.5.
2. The test sections that have nonplastic embankments (high strengths) are generally overdesigned according to these methods. The field performance confirms this because none of those sections are below a PSI of 3.0. Based on this fact, plus the fact that the life predictions are on the order of 40 to 50 years beyond 1967, it has been recommended that a reduction in thickness of pavements on these embankments be made on a trial basis.
3. Some of the sections on plastic soils appear to be overdesigned, as indicated by the long predicted lives, and some others appear to be underdesigned.
4. The parameters necessary to use these design methods can be evaluated in Minnesota and could be used to design flexible pavements in Minnesota.
5. For test sections with granular bases and subbases it is necessary to provide adequate drainage. The three sections that had much less life than would be predicted by these design procedures had poor drainage characteristics primarily because of trench-type constructions. On the other hand, two of the sections that are performing better than anticipated both have good drainage characteristics.
6. Even though the Asphalt Institute design procedure is based on thicknesses that are conservative compared to the AASHO Road Test data, rather than going through the middle of the data, as do the equations of the AASHO Interim Guide, the life predictions using the Asphalt Institute procedures are less conservative (longer). This happens because different types of equations or models have been fitted through the Road Test data.
7. Observation of the test sections should be continued at least until they reach a PSI of 2.5. This can be done adequately by determining the PSI once per year on each test section. The traffic can be updated each year using the statewide traffic report and the traffic program.

Four equations were developed relating Benkelman beam deflection to performance at the AASHO Road Test. The equations relate the number of applications of a given load to a PSI of 2.5 and 1.5 to the fall and spring deflections. The equation for spring deflections predicting equivalent 18,000-lb applications to a PSI of 2.5 is shown as Eq. 4. The equation using spring deflections is considered because the correlation is better than for fall deflections and the standard error is less. This is reasonable because the spring period is the most critical time in Illinois just as in Minnesota.

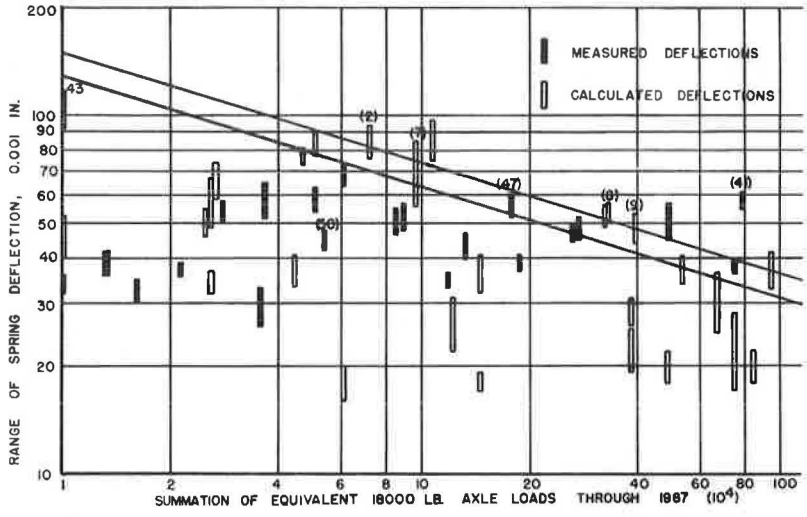

Figure 1. Measured and predicted spring deflection related to equivalent 18,000-lb axle loads sustained by the test section through 1967.

$$\log (\Sigma N18)_{2.5} = 11.06 - 3.25 \log d_S \tag{4}$$

Squared correlation coefficient = 0.78
Standard error = 0.21

where

$(\Sigma N18)_{2.5}$ = summation of equivalent 18,000-lb axle loads to a PSI level of 2.5; and
d_S = Benkelman beam deflection taken during the spring period, 0.001 in.

At the AASHO Road Test, the normal deflection procedure yielded about the same deflections as the rebound procedure. The rebound procedure is used for determining deflection in Minnesota because this method yields a more accurate deflection on weak pavements. The deflection term in Eq. 4 is therefore assumed to be comparable to rebound deflections measured in Minnesota.

To see how the Road Test equation worked for Minnesota pavement strengths and mixed traffic, plots such as Figure 1 were made. The upper line on the figure is Eq. 4; the lower line represents one standard error conservative from this best-fit line. For each of the test sections a maximum spring deflection level has either been measured directly or estimated from fall deflections for the spring of 1964, 1965, 1966, and 1967. The range of these deflections for the four springs for each of the sections is plotted against the summation of equivalent 18,000-lb axle loads the section has sustained from the time of construction to 1967. The spring deflections have been adjusted to account for the maximum allowable spring axle loads on the test section. In 1967 all of the sections had serviceability levels higher than 2.5 except TS 28, 9, 41, and 50. Test Sections 2, 8, and 41 have been overlaid. In addition to these, only TS 7, 43, and 47 had serviceability levels less than 3.0. The other sections are performing well through 1967 and therefore for the ΣN18 values indicated for the section on the plot.

The position of the points checks Eq. 4 to some degree at this time because most of the points are to the left of the best-fit line. This means that the equation predicts that the sections can generally withstand more applications of load than have been sustained through 1967.

Unless a test section is overlaid in 1968, it will move to the right on the figure by the increase in log (ΣN18) represented by the 1968 traffic. If observations of serviceability and determinations of traffic are made in future years until each section reaches

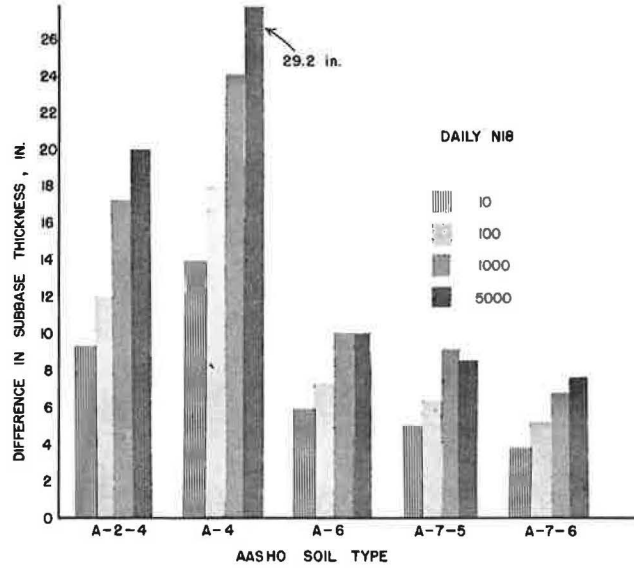

Figure 2. Differences in subbase thickness represented by variation in embankment strength for the AASHO soil classes using the AASHO Interim Guide R-value design procedure.

a serviceability level of 2.5, then a performance line for the Minnesota sections could be established. It may or may not agree with Eq. 4 from the Road Test.

Based on the position of the points in Figure 1 it has been recommended that a line one standard error conservative from the best-fit line from the Road Test be used for the present. This can be represented by Eq. 5, with terms as defined for Eq. 4:

$$\log (\Sigma N18)_{2.5} = 10.85 - 3.25 \log d_s \tag{5}$$

With additional observations on sections in the future, this recommendation could be changed.

The advantage of using a strength test such as the Benkelman beam deflection to predict performance is that all localized conditions such as drainage, poor quality base material, and the like are evaluated. Using only thickness as an evaluation, this is not possible. These performance predictions with the deflection test are used to develop an overlay design procedure. The deflection test has also been established as a good method for determining allowable maximum spring axle loads.

EVALUATION OF THE PRESENT MINNESOTA FLEXIBLE PAVEMENT DESIGN METHOD

Based on the results of the performance studies on the Minnesota test sections, certain modifications to the present Minnesota design procedure are suggested. Some of these are direct modifications, such as using a strength test for embankment evaluation and changes in thickness requirements, whereas others are relationships developed from the data but that are not presently considered directly in the present procedure, such as an overlay design. In this section the recommendations are outlined and justified.

Embankment Evaluation

At the present time the AASHO classification system is used to evaluate embankments for design in Minnesota. "Standard" sections are given for an A-6 soil and are selected on the basis of the total volume of traffic (ADT) and the volume of heavy com-

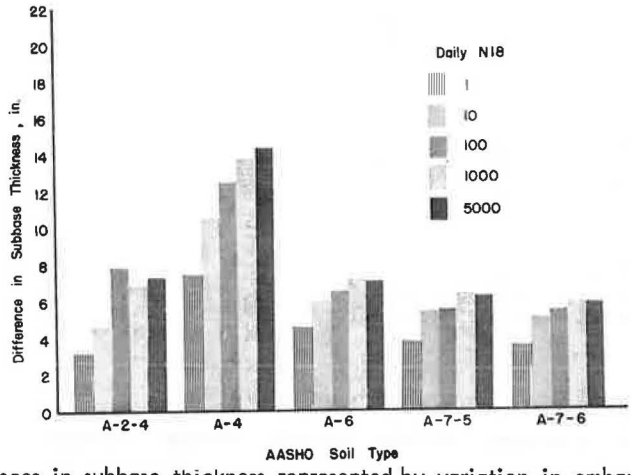

Figure 3. Differences in subbase thickness represented by variation in embankment strength for the AASHO soil classes using the Asphalt Institute R-value design procedure.

mercial trucks (HCADT). It is recommended that strength be used as a criterion for embankment classification based on the experience of a number of other agencies, such as the California Division of Highways (19), the Corps of Engineers (20), the Asphalt Institute (3) and others. For the Minnesota design procedure the recommended thicknesses are constant except for the A-1, A-2, and A-4 soils. However, the variation in strength found within each AASHO classification represents a range in pavement design thicknesses based on design procedures that consider the strength of the embankment. Figure 2 is a plot of the variation in subbase thickness that results if the AASHO Interim Guide is used for design on the weakest and strongest soils according to the R-values found within each AASHO classification. Figure 2 shows that the difference in design thickness in terms of granular subbase ranges from about 5 in. to 29 in., depending on the embankment soil type and the level of traffic. Figure 3 shows the differences in subbase thickness represented by the variation in R-value based on the Asphalt Institute design procedure. These plots show that, even though the AASHO classification generally denotes the strength of a soil, within a given class the variation in strength can be significant. The variation in thickness is shown to be most significant for A-2-4 soils followed by the A-6, A-7-5, and A-7-6 soils in that order. The differences are greatest using the AASHO Guide procedure compared to the Asphalt Institute procedure.

Correlations have been made between the field E-modulus from the plate load test and both the R-value and CBR. For the R-value, correlations were made for R-values obtained at exudation pressures of 100, 200, 240, 300, and 400 psi. The embankment E-moduli have been determined at 5-, 15-, and 25-psi plate pressures. The correlations were best for the R-values relative to the CBR and for the R-value determined at an exudation pressure of 200 psi. Based on these correlations plus the fact that the stabilometer R-value is somewhat easier to run than the laboratory CBR test, the stabilometer R-value has been recommend for evaluating embankment soils in Minnesota.

Evaluation of Materials in the Pavement Structure

The present Minnesota design method uses the gravel equivalent concept to evaluate the relative effect of the pavement layers for design. Ultimately, the relative effect of the layers should be based on the ability of the various layers to improve the performance of a pavement section. The equivalencies from the AASHO Road Test are the only ones based on performance at the present time. The relative effect of the layers has also been determined based on deflections that have also been related to performance. For the Minnesota test sections plus the sections used for Minnesota Investi-

gation 603 (22), equivalencies have been roughly implied based on spring tonnage obtained and deflections. The elastic theory has also been used to predict deflections using appropriate moduli for the layers. The problem with determining equivalencies is that, with variations in field conditions, the equivalencies can vary significantly on a daily and even hourly basis with changes in temperature, moisture content, etc.

Using an equivalency factor of 1.0 for granular-base materials, plant-mix surface equivalency factors have been shown to range from less than 1.0 to 15 depending on the method of evaluation and conditions assumed for evaluation. The greatest percentage of values is in the 2.2 to 3.5 range for critical field conditions. At this time it has therefore been recommended that use of the present factor of 2.25 for hot-mix asphalt concrete surfacing (mixes 2351 and 2341) be continued. A value of 2.00 can be used for the 2331 mix, which has a somewhat more open aggregate gradation requirement. It has also been recommended that the present factors be used for stabilized base courses. These are 1.50 for rich-mixed 2204 mixes and 1.25 for lean bituminous-treated bases. These values compare favorably with the factors determined by Terrel and Monismith for cured mixes using emulsions (23).

It has also been recommended that an equivalency of 1.00 be applied to subbase materials that fail base course specifications only because of material retained on the 1-in. sieve. Also, based on the low R-values found for the base materials on TS 28 and 29, it has been recommended that the gradation and degradation properties of crushed limestone bases be checked carefully. As more performance data are obtained on the test sections it will be possible to determine appropriate equivalencies based on this factor.

Traffic Evaluation

As a result of the study of traffic in this investigation, it has been recommended that the traffic factor in the design method be based on equivalent 18,000-lb loads. With such a factor, changes in the composition of the truck population can be more accurately handled than with a term such as HCADT. It will be possible to take advantage of the trends shown in the last few years that indicate an increase in equivalent 18,000-lb axle loads for a given number of trucks.

Thickness Recommendations

Although the histories of serviceability ratings on the Minnesota sections are still relatively short, it has been possible to make predictions of life by the models discussed and with the Benkelman beam spring deflections. These predictions permit, in effect, a judgment of the present Minnesota design. The results have been studied for various embankment types, R-values of the embankment, and traffic categories. Generally it has been found that the present Minnesota procedure is somewhat conservative for sections with relatively strong embankments. Recommendations are being made for changes in gravel equivalent thicknesses from minus 8 in. to plus 2 in. Continuing observations on the test sections may alter or strengthen these recommendations.

Designing an Asphalt Overlay Using Benkelman Beam Deflections

Benkelman beam deflections have been related to the performance of an asphalt pavement using Eq. 5. Equation 6 was developed for plastic embankment soils from data from a deflection study on Minnesota secondary roads (22) in addition to the data from this satellite study:

$$\log d_S = 2.300 - 0.051\, D_1 - 0.022\, D_2 \qquad (6)$$

where

d_S = spring Benkelman beam deflection, 0.001 in.;
D_1 = asphalt mixture thickness, in.; and
D_2 = granular base plus subbase thickness, in.

With the logarithmic relationship for deflection, the antilogs of the coefficients indicate a percentage reduction in deflection per inch of material. It is stated that this equation should not be used as a design equation (22), but considering all other conditions constant it can be assumed that each inch of asphalt mixture decreases the deflection by about 10 to 12 percent. Using the traffic information developed in this study, it is possible to relate present traffic in terms of a daily number of equivalent loads or

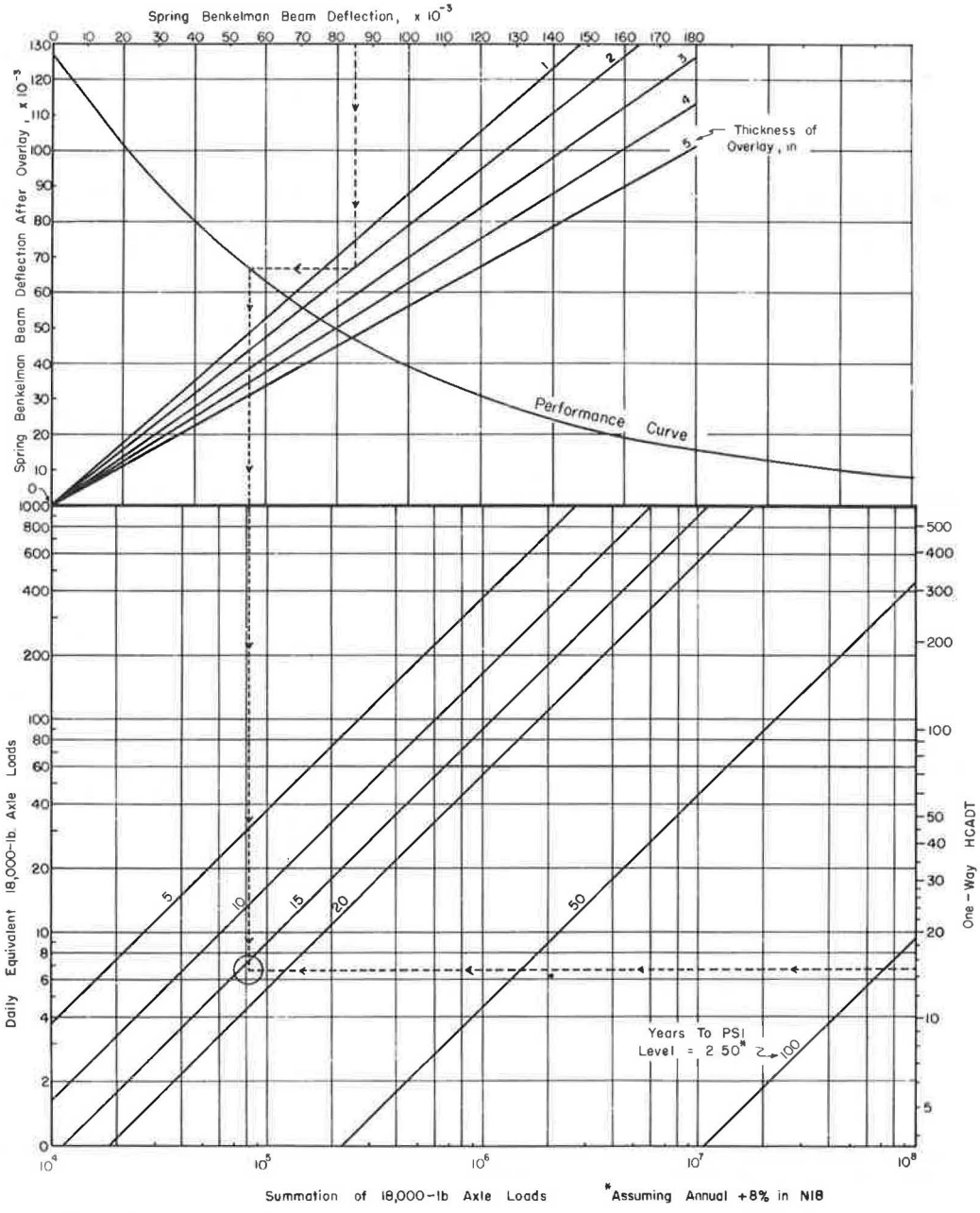

Figure 4. Performance predictions for pavements to be overlaid based on spring Benkelman beam deflections before overlay (for plastic embankments).

HCADT to the number of years it would take to accumulate a given total number of equivalent 18,000-lb axle loads. The number of years to accumulate the loads is, of course, dependent on the annual rate of increase in traffic assumed.

When these relationships are combined, a method such as that shown in Figure 4 could be developed to help estimate the effectiveness of an overlay in extending the life of an asphalt pavement. The upper part of the chart includes both the relationship between spring deflection before overlay and what it can be expected to be after various overlay thicknesses are added to the pavement section, and then how many equivalent 18,000-lb axle loads the pavement can be expected to withstand before the PSI level drops to 2.5 (assuming that the pavement is constructed to a PSI of about 4.0). The total number of equivalent loads is on the bottom axis of the chart. The lower part of the chart shows the relationship between present daily N18, HCADT, and how many years it would take to accumulate the given total number of equivalent axle loads.

The chart is set up for an annual equivalent load increase of 8 percent, but it could easily be modified to another value. The other relationships can also be changed as more information becomes available. It is felt that the present chart is somewhat conservative, which is necessary until more performance data are available from the Minnesota test sections. It is recommended that as many checks as possible be made on the relationships shown in the chart, especially the decrease in deflection observed on a pavement section after it has been overlaid. If possible, some test areas should be established where, on a given job, some ½-mile sections could be overlaid with 1 in. of asphalt, some sections overlaid with 2 in., and so forth, so that both the decrease in deflection and the relative performance of the sections could be observed. This additional information could then be added to the overall system of evaluation of flexible pavements.

This method could also be used as an indicator of an appropriate maximum spring axle load. For instance, if the life of a given overlay was considered too short, the spring deflection could be decreased by limiting the axle load on the pavement during that period. Thus, if a 7-ton axle load were the maximum load rather than a 9-ton load, the spring deflection could be reduced by seven-ninths with a corresponding increase in predicted life of the overlay section.

SUMMARY

By establishing 50 test sections on in-service roads in Minnesota and following a program of measurements—including serviceability determinations, plate tests, Benkelman beam deflections, and traffic—an evaluation of flexible pavements is being made using concepts originally developed at the AASHO Road Test.

A correlation has been developed so that present serviceability indexes can be determined using the Minnesota roughometer. Traffic is being evaluated on the basis of equivalent 18,000-lb axle loads. The performance of the Minnesota test sections for four years has been determined and methods of extending the performance trends are given. A method of utilizing spring Benkelman beam deflections to predict performance shows promise.

Modifications to the present Minnesota flexible pavement design are to be considered on the basis of these findings. Continued observation of the test sections is needed to verify the indicated relationships.

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