# COMPARISON STUDY ON THE PERFORMANCE OF BITUMINOUS STABILIZED BASES (M 66 AND M 20)

MICHIGAN DEPARTMENT OF STATE HIGHWAYS AND TRANSPORTATION

TESTING AND RESEARCH DIVISION
RESEARCH LABORATORY SECTION

# COMPARISON STUDY ON THE PERFORMANCE OF BITUMINOUS STABILIZED BASES (M 66 AND M 20)

F. T. Hsia

Research Laboratory Section
Testing and Research Division
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## Introduction

In a letter dated September 30, 1975, K. A. Allemeier, Engineer of Testing and Research, requested that the Research Laboratory conduct a comparative pavement performance study, based on Benkelman beam readings, of two different pavement cross-sections. The main objective of this study is to determine if there is any significant difference in the strength of the two base designs.

Cross-section I, with the bituminous concrete pavement constructed on 11 in. of aggregate base course and 18 in. of subbase, represents an 11-mile section of M 20. Cross-section II, a bituminous concrete pavement constructed on a 4.5-in. bituminous stabilized base course (commonly referred to as black base) with 4 in. of selected aggregate subbase and 18 in. of subbase, represents an eight-mile section of M 66. Both sections have approximately the same traffic volumes, soil conditions, climate, and completion date.

In addition to Benkelman beam deflection comparisons, the two cross-sections were also compared on the basis of allowable springtime loads which were derived directly from Benkelman beam readings.

## Description of Test Area

The black base section, shown in Figure 1A, consists of 2.25 in. bituminous concrete, 4 in. of black base, 4 in. of selected aggregate base, and 18 in. of subbase representing an eight-mile section of M 66 from Stanton north to M 46. The comparable aggregate base section, shown in Figure 1B, consists of 2.75 in. of bituminous concrete, 11 in. of aggregate base and 18 in. of subbase representing an 11-mile section of M 20 from Remus, in Mecosta County, east to Gilmore Rd in Isabella County. Descriptions of the bituminous surface layer of both sections are summarized in Table 1. Both sections have approximately the same traffic volumes, soil conditions, climate, and completion date. Three comparable sites, with lengths ranging from 1,000 ft to 2,500 ft and 10 test points for Benkelman beam measurements distributed evenly over the length of each site, were selected for each base section (Table 2). A general layout of test sites and locations of frost depth indicators used for checking the spring thaw condition are illustrated in Figure 2.

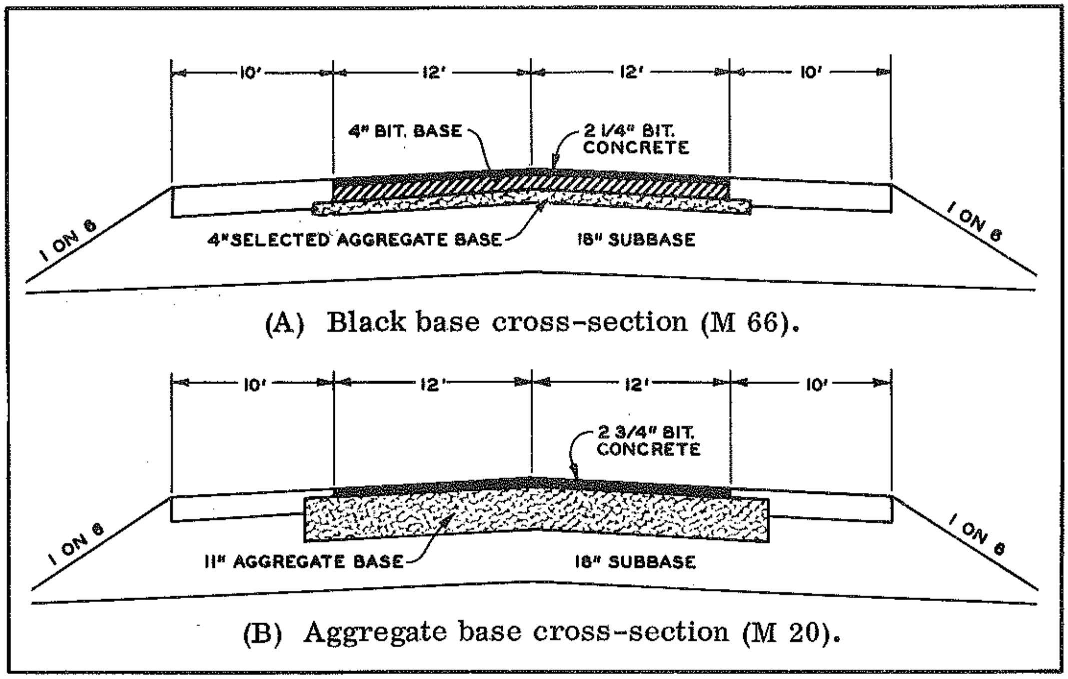



Figure 1. Cross-sections of the two bases.

TABLE 1
BITUMINOUS SURFACE LAYERS

| | Item | Rate of Application | Estimated Thickness, in. | Asphalt Penetration |
|---|---|---|---|---|
| Black Base | Bituminous Concrete Wearing Course, Type M | 100 lb/sq yd | 1 | 85-100 |
| Black Base | Bituminous Bond Coat | 0.05 gal/sq yd | | |
| Black Base | Bituminous Concrete Leveling Course, 25A | 150 lb/sq yd | 1-1/4 | 85-100 |
| Black Base | Bituminous Bond Coat | 0.05 gal/sq yd | | |
| Aggregate Base | Bituminous Binder Course | 180 lb/sq yd | 1-3/4 | 120-150 |
| Aggregate Base | Bituminous Concrete Wearing Course, Type M | 100 lb/sq yd | 1 | 120-150 |
| Aggregate Base | Bituminous Prime Coat | 0.30 gal/sq yd | | |
| Aggregate Base | Bituminous Bond Coat | 0.05 gal/sq yd | | |

TABLE 2
GENERAL TEST SITE DESCRIPTION
(Both completed, October 1974)

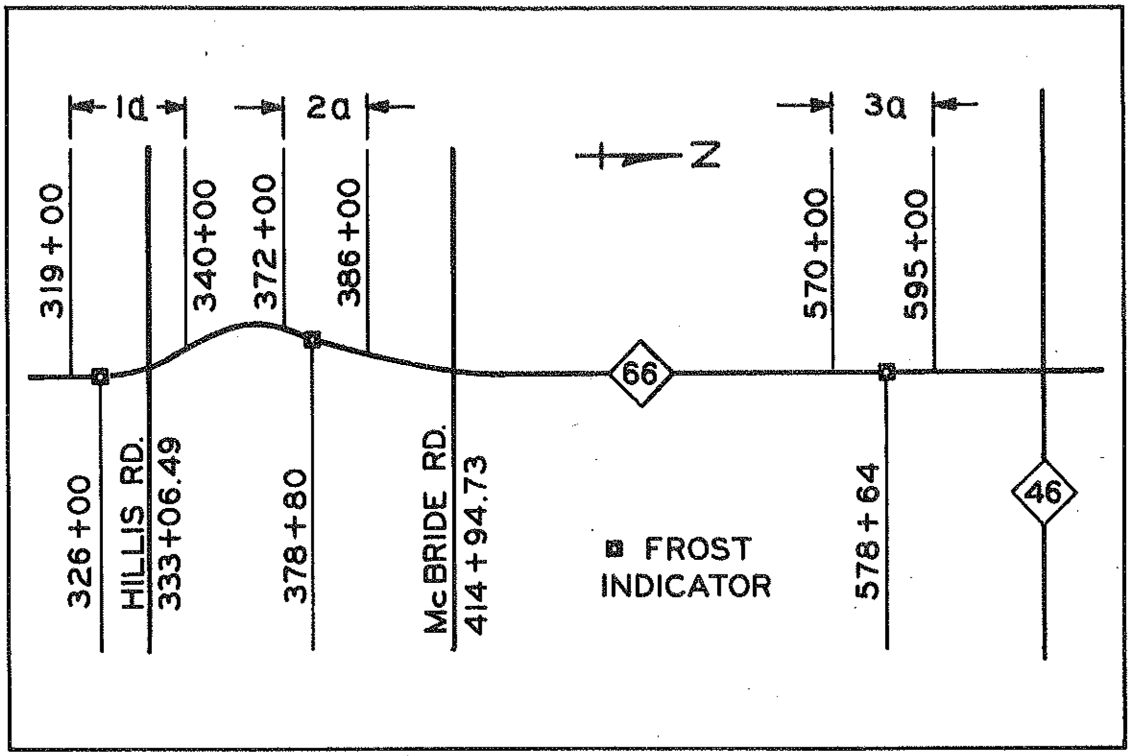
| Subgrade | Black Base Section | | | Aggregate Base Section | | |
|---|---|---|---|---|---|---|
| | Site No. | Station | Traffic | Site No. | Station | Traffic |
| Berrien and Coral - shallow cut and fill | 1a | 319+00 to 340+00 | ADT: 22 - 2700, 7% commercial | 1b | 30+00 to 50+00 | ADT: 22 - 2600 4.5% commercial |
| McBride - cut | 2a | 372+00 to 386+00 | | 2b | 184+00 to 194+00 | |
| McBride and Coral - fill | 3a | 570+00 to 595+00 | | 3b | 205+00 to 230+00 | |

## Testing Procedure

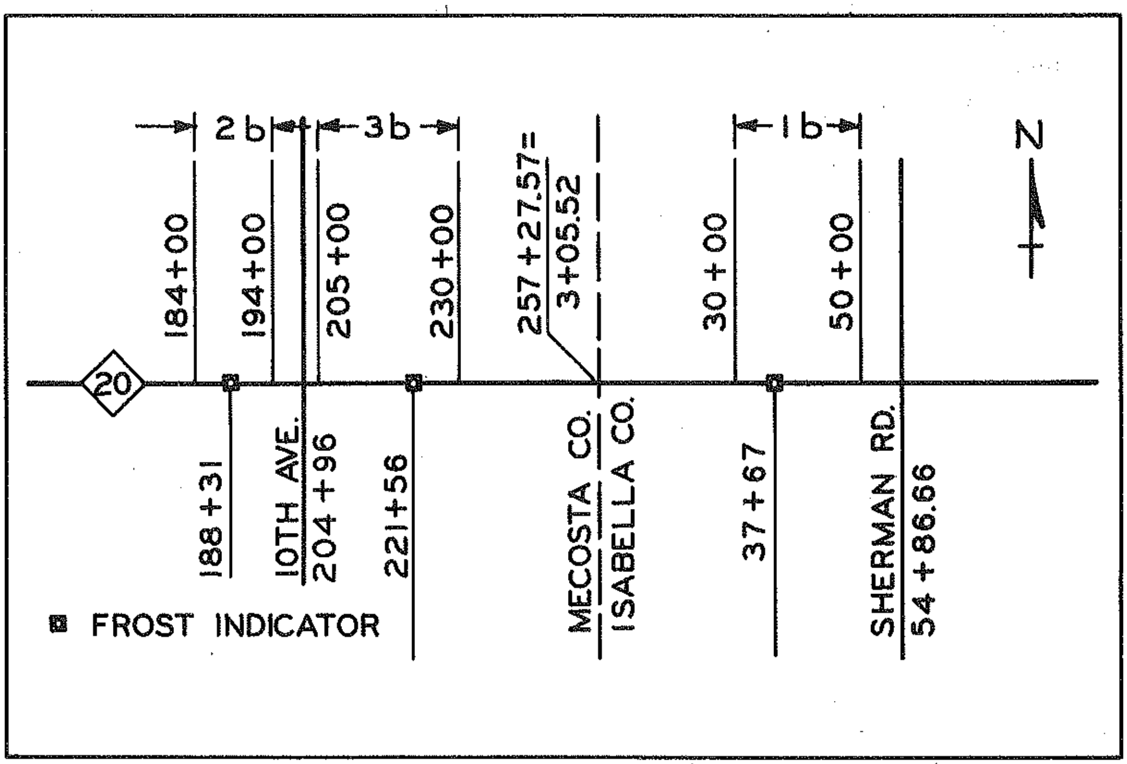
A rebound procedure developed by the Canadian Good Roads Association (CGRA) was used in this project (1). In this method, a standard 18-kip axle load truck, with a tire pressure of 70 psi, serves as the loading system. Deflection is initially recorded when the probe is located between the tires (Fig. 3); an intermediate reading is taken when the truck is moved 8 ft - 10 in. from the probe (Fig. 4), and a final reading with the load truck at least 30 ft away. According to Ref. (1), apparent rebound measurements can be recognized by comparing the intermediate and final readings. If a differential of more than 0.001 in. exists, the reading must be corrected to determine the actual rebound values by means of a formula provided by CGRA. However, intermediate readings and final readings were found to be almost identical in this project. Therefore, pavement deflections were taken as the differences between the initial readings and the final readings.

As recommended by CGRA, the point of deflection measurement was set at a distance of approximately 3 ft from the edge of the pavement. Temperature measurements were made for each test point (Fig. 5) and deflections were corrected to 80 F values by using Table 3 as recommended by Ref. (2).

During the year six measurements were made at each site, four in the spring and two in the summer. Measurements for comparable sites were




(A) Black base sites (M 66, C/S 59051, J/N 01770A).




(B) Aggregate base sites (M 20, C/S 54022 and 37021, J/N 00519A and 05101A).

Figure 2. Site layout.

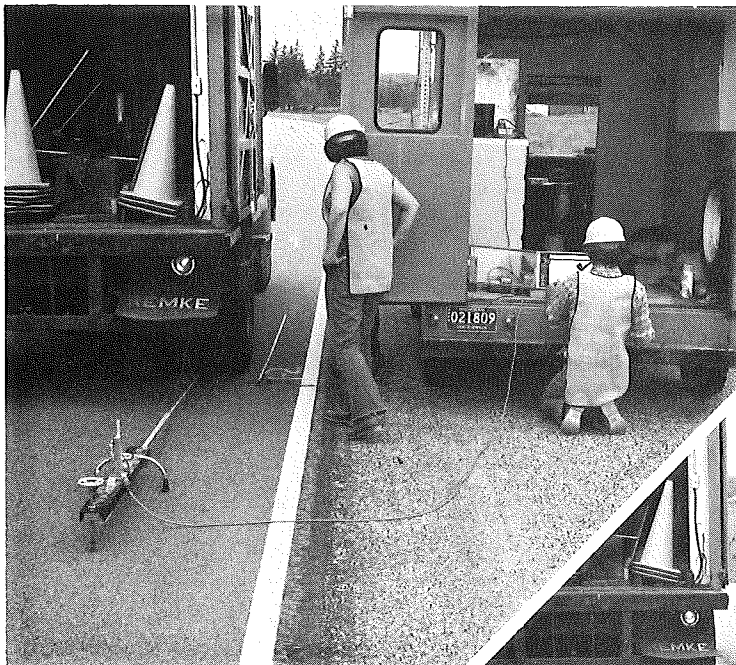



Figure 3. Recording the initial deflection with probe between tires.

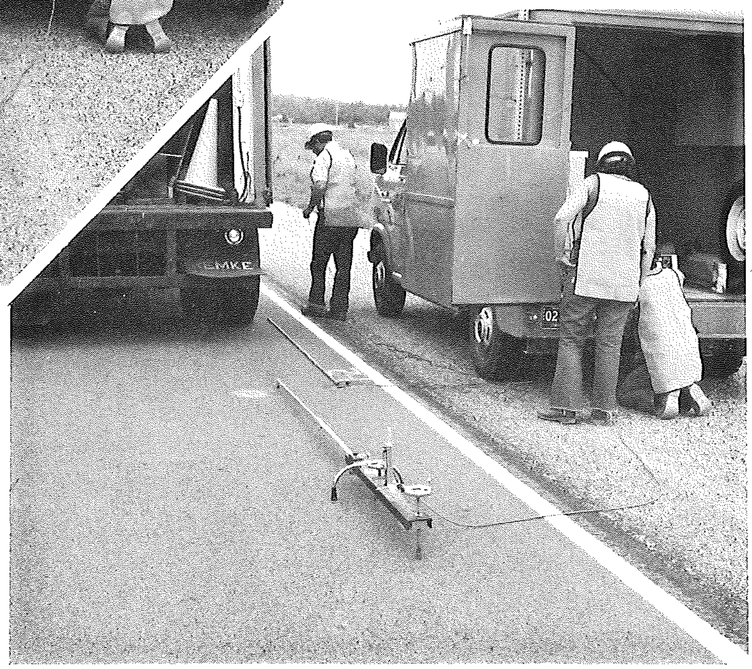
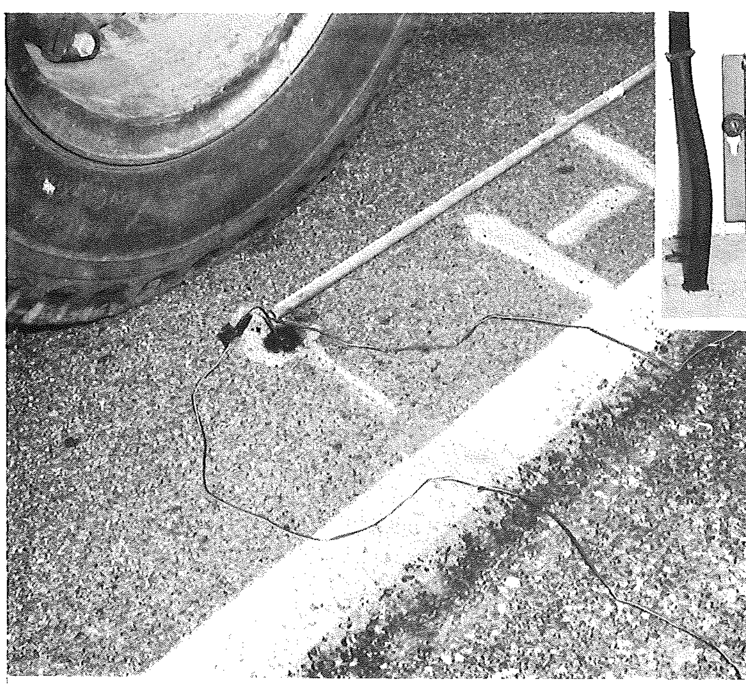


Figure 4. Recording deflections while the load truck is creeping forward.



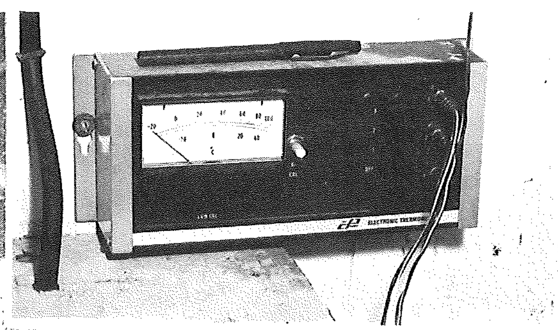


Figure 5. Oil-filled hole for pavement temperature measurements. Temperature recorder above.

TABLE 3
BENKELMAN BEAM DEFLECTION CORRECTIONS TO 80 F
(After Minnesota's study (2))

| Deflection Range, in. | Temperature, F | | | | |
|---|---|---|---|---|---|
| | to 35 | 36 - 45 | 46 - 55 | 56 - 65 | 66 - 75 |
| 0.000 - 0.010 | 0.005 | 0.004 | 0.003 | 0.002 | 0.001 |
| 0.010 - 0.020 | 0.007 | 0.006 | 0.004 | 0.003 | 0.001 |
| 0.020 - 0.030 | 0.010 | 0.008 | 0.006 | 0.004 | 0.002 |
| 0.030 - 0.040 | 0.010 | 0.008 | 0.006 | 0.004 | 0.002 |
| 0.040 - 0.050 | 0.012 | 0.010 | 0.007 | 0.005 | 0.002 |
| 0.050 - 0.060 | 0.015 | 0.012 | 0.009 | 0.006 | 0.003 |

All corrections to be added.

NOTE: For deflections over 0.060 in. no data have, as yet, been obtained. It is suggested that the corrections for 0.050 to 0.060 in. deflections be used for higher deflections.

made during an interval of no more than one day to ensure no drastic change in environmental conditions.

Test Results

Benkelman beam measurement data were recorded in columns 1 through 9 of Tables 4 and 5 for the black base and aggregate base sections. In these tables:

BB = individual deflections for each point; there are 10 points in each site.

$\overline{BB}$ = arithmetic average of the 10 individual deflections.

s = standard deviation = $\sqrt{(BB - \overline{BB})^2 / (n-1)}$, where n = 10

$\overline{BB}_{80}$ = $\overline{BB}$ at 80 F, converted by Table 3.

Mean $\overline{BB}_{80}$ = arithmetic average of the six values measured at different times for each site.

# TABLE 4

## DEFLECTION DATA AND ALLOWABLE SPRINGTIME AXLE LOAD DETERMINATIONS BLACK BASE PAVEMENT (M 66; ADT = 22 - 2700)

| Site No. | Subgrade | Surface Thick-ness, in. | Deflections for each point, BB ($10^{-3}$ in.) | Pave-ment Temp, F. | Date | Average Deflection, $\overline{BB}$ ($10^{-3}$ in.) | Standard Deviation, s ($10^{-3}$ in.) | Average Deflection at 80 F, $\overline{BB}_{80}$ ($10^{-3}$ in.) | Present Design Deflection, $\overline{BB}_{80}$ + 2s ($10^{-3}$ in.) | Spring-time Ratio | Design Springtime Deflection, SBB ($10^{-3}$ in.) | Allowable Springtime Deflection, ABB ($10^{-3}$ in.) | Allowable Springtime Load, $L_A$, tons |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1a | Sandy clay - cut and fill (Berrien and Coral) | 2.25 | 9,11,10,15,13,11,10,11,12,11 | 52 | 3-24-76 | 11.3 | 1.7 | 15.3 | 18.7 | 1.0 | 18.7 | 60 | 28.9 |
| | | | 10,15,12,16,12,10,10,12,14,11 | 57 | 3-30-76 | 12.1 | 2.1 | 15.1 | 19.3 | 1.0 | 19.3 | 60 | 28.0 |
| | | | 10,10,11,13,11,10,11,12,12,11 | 55 | 4- 6-76 | 11.1 | 1.0 | 15.1 | 17.1 | 1.0 | 17.1 | 60 | 31.6 |
| | | | 8,8,9,12,11,9,8,10,12,8 | 45 | 4-27-76 | 9.5 | 1.6 | 13.5 | 16.7 | 1.0 | 16.7 | 60 | 32.3 |
| | | | 10,11,11,14,12,11,14,16,13,13 | 97 | 6-14-76 | 12.3 | 1.7 | 12.3 | 15.7 | 1.5 | 23.6 | 60 | 22.9 |
| | | | 11,12,12,15,12,10,11,13,13,13 | 84 | 8-10-76 | 12.1 | 1.2 | 12.1 | 14.5 | 1.75 | 25.4 | 60 | 21.3 |
| | | | | | | | Mean $\overline{BB}_{80}$ = 13.9 | | | | | | Mean $L_A$ = 27.5 |
| 2a | Sand and loam - cut (McBride) | 2.25 | 10,8,8,13,11,10,10,11,13,11 | 62 | 3-24-76 | 12.2 | 2.0 | 15.2 | 19.2 | 1.0 | 19.2 | 60 | 28.1 |
| | | | 12,9,10,14,12,11,13,12,14,13 | 61 | 3-30-76 | 12.1 | 1.7 | 15.1 | 18.5 | 1.0 | 18.5 | 60 | 29.2 |
| | | | 13,10,10,14,11,11,12,12,13,13 | 64 | 4- 6-76 | 12.0 | 1.4 | 15.0 | 17.8 | 1.0 | 17.8 | 60 | 30.3 |
| | | | 11,9,9,12,10,15,12,11,12,11 | 48 | 4-27-76 | 11.1 | 1.8 | 15.1 | 18.7 | 1.0 | 18.7 | 60 | 28.9 |
| | | | 16,11,10,16,12,11,14,12,12,11 | 107 | 6-14-76 | 12.6 | 2.2 | 12.6 | 17.0 | 1.4 | 23.8 | 60 | 22.7 |
| | | | 13,10,11,13,12,11,12,11,12,11 | 95 | 8-10-76 | 11.6 | 1.1 | 11.6 | 13.8 | 1.55 | 21.4 | 60 | 25.2 |
| | | | | | | | Mean $\overline{BB}_{80}$ = 14.1 | | | | | | Mean $L_A$ = 27.4 |
| 3a | Sand and loam - fill (McBride and Coral) | 2.25 | 15,15,15,16,16,16,11,18,17,16 | 79 | 3-24-76 | 15.5 | 1.8 | 15.5 | 19.1 | 1.0 | 19.1 | 60 | 28.3 |
| | | | 16,17,16,17,18,17,11,14,18,15 | 75 | 3-30-76 | 15.9 | 2.2 | 15.9 | 20.3 | 1.0 | 20.3 | 60 | 26.6 |
| | | | 14,15,17,16,18,18,14,11,18,17 | 79 | 4- 6-76 | 15.7 | 2.3 | 15.7 | 20.3 | 1.0 | 20.3 | 60 | 26.6 |
| | | | 13,12,12,12,12,12,9,11,12,12 | 51 | 4-27-76 | 11.6 | 1.1 | 15.6 | 17.8 | 1.0 | 17.8 | 60 | 30.3 |
| | | | 15,13,15,13,16,16,10,8,15,13 | 105 | 6-14-76 | 13.3 | 2.6 | 13.3 | 18.5 | 1.4 | 25.4 | 60 | 20.8 |
| | | | 13,13,13,11,12,13,10,12,13,13 | 99 | 8-10-76 | 12.3 | 1.1 | 12.3 | 14.5 | 1.55 | 22.5 | 60 | 24.0 |
| | | | | | | | Mean $\overline{BB}_{80}$ = 14.7 | | | | | | Mean $L_A$ = 26.1 |
| | | | | | | | | | | | | | Overall Mean $L_A$ = 27 |

# TABLE 5

## DEFLECTION DATA AND ALLOWABLE SPRINGTIME AXLE LOAD DETERMINATIONS AGGREGATE BASE PAVEMENT (M 20; ADT = 2600)

| Site No. | Subgrade | Surface Thick-ness, in. | Deflections for each point, BB ($10^{-3}$ in.) | Pave-ment Temp, F. | Date | Average Deflection, $\overline{BB}$ ($10^{-3}$ in.) | Standard Deviation, s ($10^{-3}$ in.) | Average Deflection at 80 F, $\overline{BB}_{80}$ ($10^{-3}$ in.) | Present Design Deflection, $\overline{BB}_{80} + 2s$ ($10^{-3}$ in.) | Spring-time Ratio | Design Springtime Deflection, SBB ($10^{-3}$ in.) | Allowable Springtime Deflection, ABB ($10^{-3}$ in.) | Allowable Springtime Load, $L_A$, tons |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1b | Sandy Clay - cut and fill | 2.75 | 16,16,18,15,16,18,18,22,15,14 | 66 | 3-23-76 | 16.7 | 2.2 | 17.7 | 22.1 | 1.0 | 22.1 | 60 | 24.4 |
| | | | 14,15,15,13,15,17,16,19,13,12 | 49 | 3-31-76 | 15.0 | 2.1 | 19.0 | 23.2 | 1.0 | 23.2 | 60 | 23.3 |
| | | | 15,14,17,13,12,17,17,17,15,13 | 74 | 4- 7-76 | 14.9 | 1.8 | 15.9 | 19.5 | 1.0 | 19.5 | 60 | 27.7 |
| | | | 17,16,18,16,17,18,17,21,15,15 | 68 | 4-27-76 | 17.0 | 1.7 | 18.0 | 21.4 | 1.0 | 21.4 | 60 | 25.2 |
| | | | 9,12,13,9,9,13,13,14,10,10 | 93 | 6-15-76 | 10.9 | 1.9 | 10.9 | 14.7 | 1.5 | 22.1 | 60 | 24.5 |
| | | | 11,11,12,10,11,13,13,11,11,9 | 104 | 8-11-76 | 11.2 | 1.4 | 11.2 | 14.0 | 1.75 | 24.5 | 60 | 22.0 |
| | | | | | | | Mean $\overline{BB}_{80}$ = 15.5 | | | | | | Mean $L_A$ = 24.5 |
| 2b | Sandy Loam - cut | 2.75 | 13,12,11,11,12,12,12,11,15,13 | 50 | 3-23-76 | 12.3 | 1.3 | 16.3 | 18.9 | 1.0 | 18.9 | 60 | 28.6 |
| | | | 12,11,11,11,11,11,11,11,16,15 | 47 | 3-31-76 | 12.1 | 1.8 | 16.1 | 19.7 | 1.0 | 19.7 | 60 | 27.4 |
| | | | 14,11,11,11,12,12,13,14,17,15 | 57 | 4- 7-76 | 13.1 | 2.0 | 16.1 | 20.1 | 1.0 | 20.1 | 60 | 26.9 |
| | | | 14,13,12,12,13,12,13,13,18,16 | 59 | 4-27-76 | 13.6 | 1.9 | 16.6 | 20.4 | 1.0 | 20.4 | 60 | 26.5 |
| | | | 14,11,12,11,12,14,13,13,16,17 | 83 | 6-15-76 | 13.3 | 2.0 | 13.3 | 17.3 | 1.4 | 24.2 | 60 | 22.3 |
| | | | 13,11,11,12,12,12,11,12,13,13 | 89 | 8-11-76 | 12.0 | 0.7 | 12.0 | 13.4 | 1.55 | 20.8 | 60 | 26.0 |
| | | | | | | | Mean $\overline{BB}_{80}$ = 15.1 | | | | | | Mean $L_A$ = 26.2 |
| 3b | Sandy Loam - fill | 2.75 | 17,18,22,17,22,23,22,23,24,22 | 55 | 3-23-76 | 21.0 | 2.7 | 27.0 | 32.4 | 1.0 | 32.4 | 60 | 16.7 |
| | | | 15,17,18,16,18,21,20,21,21,19 | 48 | 3-31-76 | 18.6 | 2.2 | 22.6 | 27.0 | 1.0 | 27.0 | 60 | 20.0 |
| | | | 15,17,19,16,18,21,20,21,22,20 | 62 | 4- 7-76 | 19.0 | 2.4 | 22.0 | 26.8 | 1.0 | 26.8 | 60 | 20.1 |
| | | | 15,17,19,15,20,21,21,22,22,20 | 62 | 4-27-76 | 19.3 | 2.6 | 22.3 | 27.5 | 1.0 | 27.5 | 60 | 19.6 |
| | | | 15,16,16,15,14,17,16,15,17,16 | 87 | 6-15-76 | 15.5 | 1.0 | 15.5 | 17.5 | 1.4 | 24.5 | 60 | 22.0 |
| | | | 12,11,14,13,14,12,15,14,14,14 | 99 | 8-11-76 | 13.1 | 1.2 | 13.1 | 15.5 | 1.55 | 24.0 | 60 | 22.5 |
| | | | | | | | Mean $\overline{BB}_{80}$ = 20.4 | | | | | | Mean $L_A$ = 20.15 |
| | | | | | | | | | | | | | Overall Mean $L_A$ = 23.62 |

## Allowable Springtime Load

Benkelman beam readings, when used as a practical means of evaluating road strength, can be related to allowable springtime loads. This relationship was developed by the Minnesota Department of Highways (3).

To determine an allowable springtime load by the Benkelman beam readings it is necessary to establish allowable deflections which can be exceeded only a relatively few times in order for the pavement to perform satisfactorily. Performance, as defined in the Minnesota procedure, is based primarily on criteria used at the AASHTO Road Test. That is, with a design period of 20 years a pavement is considered to have performed satisfactorily if at the end of 20 years its Present Serviceability Index has not dropped below a "terminal index" of 2.5. (If the allowable deflections have been accurately chosen and deflections which exceed the allowable occur repeatedly the pavement will fail, or reach the terminal index, in less than 20 years.) In addition to the AASHTO Road Test results, the Minnesota procedure also utilizes results obtained from the research done at the WASHO Road Test, by the state of California, by the Canadian Good Roads Association, and other sources. A second achievement in the Minnesota investigation was to verify the use of the recommended allowable deflections as influenced by factors such as climate, type of aggregate, type of mix, etc.

Tables 4 and 5 furnish all the measured deflection data and the allowable springtime axle load determinations obtained according to the Minnesota procedure, explained in Ref. (3) and worked out step-by-step in the example in the Appendix. Some of the terminology and their estimations, shown in Tables 4 and 5 were excerpted from Ref. (3) and are listed as follows:

$\overline{BB}_{80} + 2s$ = present design deflection

deflection ratio (springtime ratio) = ratio of springtime deflections to deflections taken during other non-frozen times of the year from Table 6 as recommended by Ref. (3).

SBB = design springtime deflection = product of present design deflection and deflection ratio.

ABB = allowable springtime deflections from Table 7 as recommended by Ref. (3).

$L_A$ = allowable springtime axle load = $L_D \frac{(ABB)}{(SBB)}$, where $L_D$ is the axle load used for deflection testing - nine tons in this project.

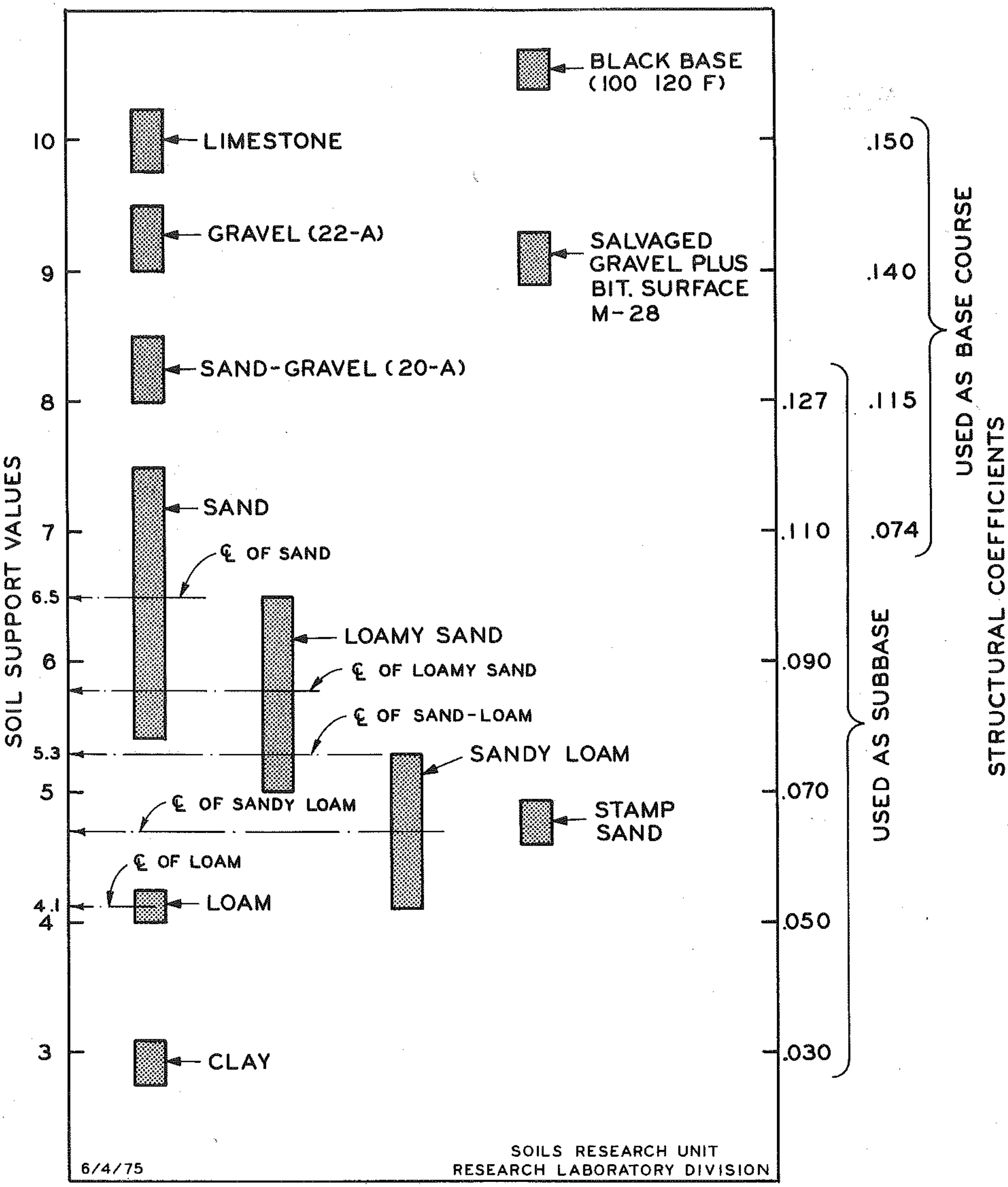



Figure 6. Range of soil support values and structural coefficients for various materials.

## TABLE 6
## DEFLECTION RATIOS USED TO CALCULATE MAXIMUM SPRINGTIME DEFLECTIONS FROM DEFLECTIONS TAKEN DURING OTHER NON-FROZEN TIMES
(After Skok (3))

| Asphalt Surface Thickness, in. | Time Of Year When Deflection Is Measured | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Sept. | 8/15 - 8-31 | 8/1 - 8/15 | 7/15 - 7/31 | 7/1 - 7/15 | 6/15 - 6/30 | 6/1 - 6/15 | 5/15 - 5/31 | 5/1 - 5/15 |
| 2.5 or less | 1.85 | 1.80 | 1.75 | 1.70 | 1.65 | 1.60 | 1.50 | 1.35 | 1.15 |
| 2.5 - 3.5 | 1.80 | 1.78 | 1.75 | 1.70 | 1.65 | 1.60 | 1.50 | 1.35 | 1.15 |
| 3.5 - 5.5 | 1.60 | 1.55 | 1.52 | 1.50 | 1.45 | 1.40 | 1.35 | 1.25 | 1.15 |
| 5.2 - 8.0 | 1.45 | 1.42 | 1.40 | 1.37 | 1.35 | 1.32 | 1.30 | 1.20 | 1.10 |
| 8 | 1.25 | 1.20 | 1.15 | 1.10 | 1.08 | 1.05 | 1.05 | 1.05 | 1.00 |

Above values are for plastic soils. For loam and silt loam embankments (slightly plastic soils) add 0.15 to tabulated values for tests run from June 15 through September and add 0.10 for tests run from May 1 through June 15.

For sand or sand and gravel embankments (non-plastic soils) 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.

Since Table 6 does not indicate how to estimate the springtime ratio of the sand-loam soils, some elaboration has to be made concerning the springtime ratio for the subgrade soil of Test Sites 3a and 3b, a sand-loam of the McBride and Coral Series. A chart of soil support values for various materials is shown in Figure 6 as described in Ref. (4). According to this figure, the average soil support values of sand, loam - sand (center line of loamy sand and sandy loam), and loam were 6.5, 5.3, 4.1, respectively. These values were related to springtime ratios, in accordance with Table 6, and Figure 7. By assuming a linear relationship between soil support values and springtime ratios, the average ratios of sand-loam for June and August were interpolated as 1.4 and 1.55, respectively.

Mean $L_A$'s shown in the last columns of Tables 4 and 5 were arithmetic mean values of the six measurements for each site.

### Discussion of Results

Figure 8 shows seasonal deflections corrected to 80 F ($\overline{BB}_{80}$) as obtained from Tables 4 and 5. It is seen that deflections during spring are generally higher than those during summer. This is believed to be due to the weaker subgrade or subbase during the spring thaw period. Deflections are practically zero during winter in northern climates where pavement and subgrades are frozen (5). Tables 4 and 5 also show that deflections in both sections are below the maximum allowable deflections as listed in Table 7.

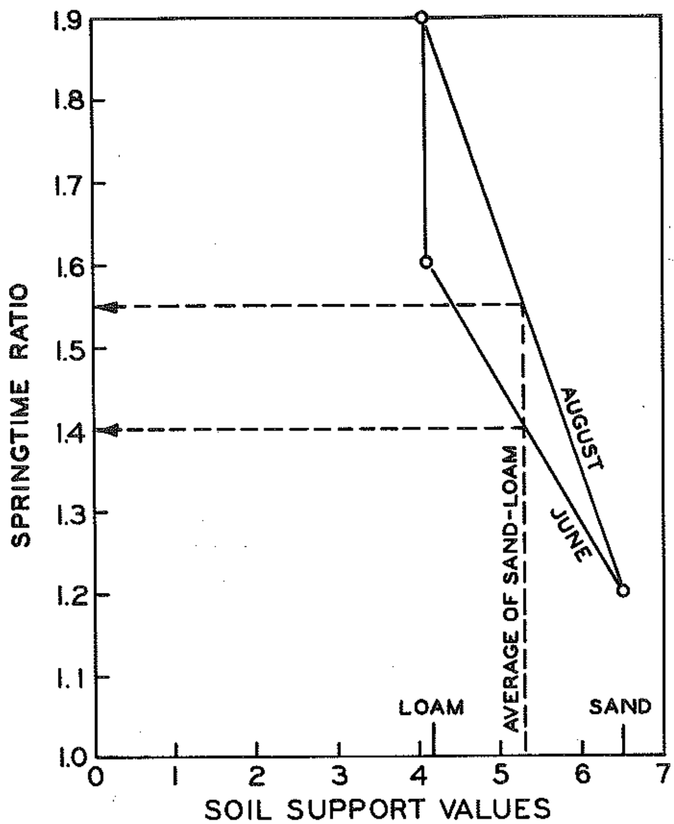



Figure 7. Linear relationship between the soil support values and springtime ratios.

Figure 8. Variation of deflections during springtime and summer.

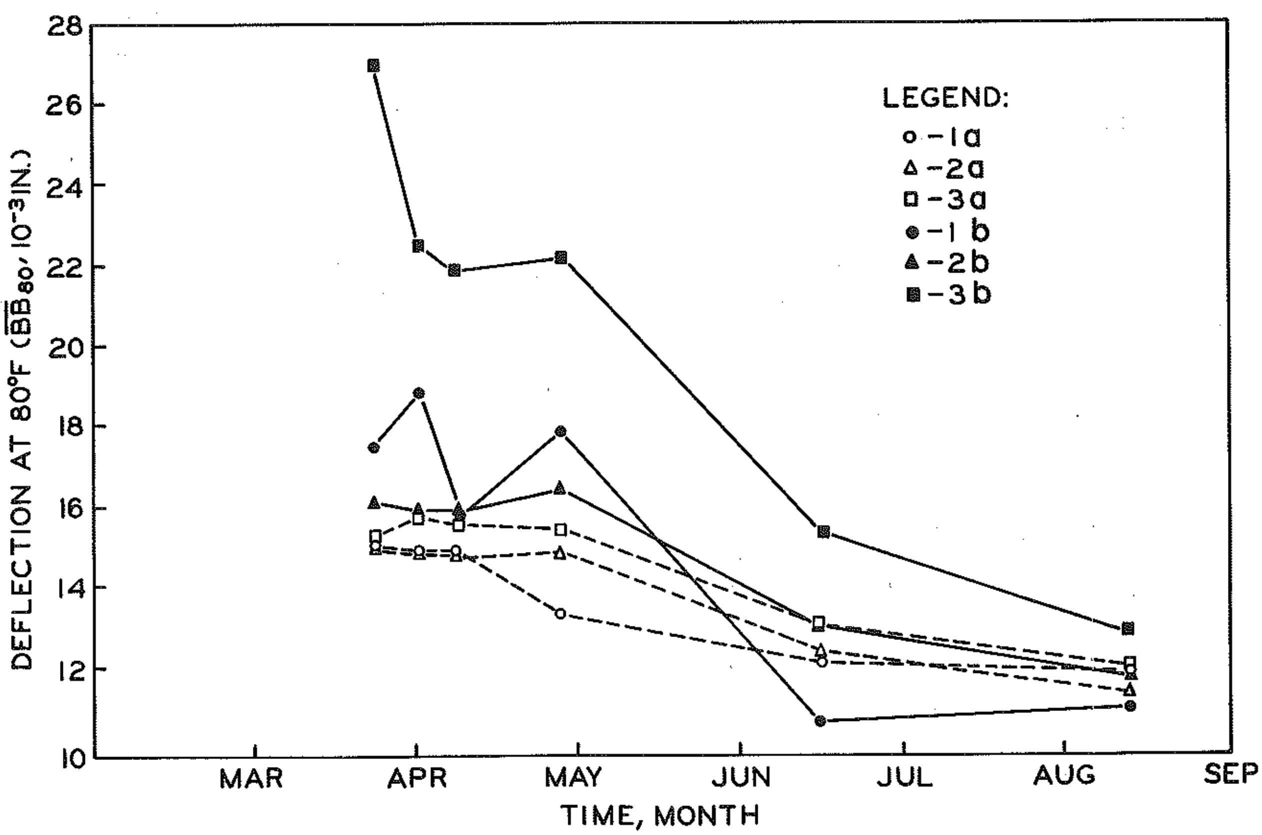


TABLE 7

ALLOWABLE SPRINGTIME DEFLECTIONS, in.

(After Skok (3))

| Traffic (two-way) | | Bituminous Thickness, in. | | |
|---|---|---|---|---|
| | | <3 | 3 - 6 | >6 |
| HCADT [1]<br>ADT [2] | <50<br><100 | 0.075 | 0.065 | 0.055 |
| HCADT<br>ADT | 50 - 100<br>500 - 1000 | 0.070 | 0.060 | 0.050 |
| HCADT<br>ADT | 100 - 150<br>1000 - 3000 | 0.060 | 0.050 | 0.040 |
| HCADT<br>ADT | >150<br>>3000 | 0.045 | 0.040 | 0.035 |

[1] HCADT = heavy commercial average daily traffic volume (excludes passenger cars and 4-tired trucks).

[2] Use ADT only when HCADT is not known.

Conclusive data from Tables 4 and 5, labeled as Mean $\overline{BB_{80}}$, were tabulated in Table 8 for comparison purposes. It is realized from Table 8 that the black base sections outperform the comparable aggregate base sections by providing lower deflection values and higher allowable springtime axle load capacities. However, the benefits are only 19.7 percent for deflection and 14 percent for springtime axle load capacity. Counting all the undetected environmental factors and possible instrument deviations, this percentage difference is considered small.

Black base pavements were not included in the Minnesota study. It should be noted, therefore, that if the black base and the asphalt concrete surface are considered as one bituminous layer, the allowable springtime deflections (ABB) in Table 4 would drop to 40 ($\times 10^{-3}$) according to Table 7. In this case, the mean allowable springtime axle load (Mean $L_A$) for 1a, 2a, 3a and the overall mean would be 18.4, 18.3, 17.4, and 18.0, respectively; less than that for the corresponding aggregate sections. Whether the black base and the asphalt concrete surface should be considered as one layer in calculating allowable springtime axle load requires further investigation which may include long-term surveys of pavement surface conditions and Benkelman beam measurements. Before such results are available, it appears suitable to consider the surface and the black base as two different layers.

TABLE 8

COMPARISON OF BLACK BASE AND AGGREGATE BASE SECTIONS

| Mean $BB_{80}$ (Deflection, $10^{-3}$ in.) | | Mean $L_A$ (Allowable springtime axle load, ton) | |
|---|---|---|---|
| Black Base | Aggregate Base | Black Base | Aggregate Base |
| 1a - 13.9 | 1b - 15.5 | 1a - 27.5 | 1b - 24.5 |
| 2a - 14.1 | 2b - 15.1 | 2a - 27.4 | 2b - 26.2 |
| 3a - 14.7 | 3b - 20.4 | 3a - 26.1 | 3b - 20.2 |
| Overall Mean - 14.2 | 17.0 | 27.0 | 23.62 |

Percent improvement using black base:

$$\frac{17 - 14.2}{14.2} \times 100 = 19.7$$

Percent improvement using black base:

$$\frac{27 - 23.62}{23.62} \times 100 = 14$$

The Benkelman beam method was introduced 30 years ago as a nondestructive test to measure the static deflection of a pavement under load, thus it does not reflect the effect of a moving load. Since then, several devices have been developed which simulate moving loads, such as the Dynaflect, road vibration machines, falling weight deflectometers, etc. It has been reported that the deflection per unit force in the Benkelman beam test is two to three times as large as that obtained with the falling weight deflectometer and that subgrade moduli derived from the former are 2.5 times smaller than that determined from the latter (6). With the recent developments in elastic layer analysis computer programs, repetitive loading tests, and field dynamic measuring devices, the Benkelman beam appears to be out of date. However, considering the expensive and sophisticated nature of the more newly developed devices, Benkelman beam readings can still be regarded as a satisfactory method for relative measurement of pavement performance and properties.

The allowable springtime axle load was computed by the Minnesota method. Although pavement properties of Michigan are different from those of Minnesota, environmental and geological conditions of both states are similar; therefore, this method is considered suitable for comparing the relative performance of Michigan pavements as done in this study. The allowable springtime deflection ABB, was estimated as 0.060 in. throughout, whereas the largest deflection measured in the field was only 0.027 in., indicating that both of the pavement sections were designed adequately.

However, deflection was the only criterion measured and it cannot be correlated to the cracking characteristics of the pavement as a whole. Other factors, such as thermal stresses, have to be considered in evaluating the adequacy of the design.

## Conclusions

Results obtained during the first year of the project indicate:

1) Benkelman beam measurements can be considered usable for determining the relative performance of the two test pavements.

2) Both black base and the aggregate base sections in this project have deflection values within allowable limits as determined by the Minnesota report.

3) The Minnesota method appears to be suitable for computing allowable springtime axle loads.

4) Black base sections were superior to the comparable aggregate base sections from the standpoint of reduced deflection and higher allowable springtime axle load capacities. However, the differences are considered minor.

5) At the present time the aggregate and the black base are both in excellent condition. Periodic observations and measurements of base and surface conditions will be made of the two sections and reports prepared as significant data are accumulated.

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# APPENDIX

## AN EXAMPLE TO ESTIMATE DEFLECTION AND SPRINGTIME LOAD-CARRYING CAPACITY AS SHOWN IN TABLES 4 AND 5

This example of calculation is taken from the second row of Table 4, i.e., deflections measured on March 24, 1976 at Site No. 1a and the allowable springtime load as calculated by the deflection data. The steps are as follows:

1. Deflections for each point, BB, are averaged to estimate the average deflection $\overline{BB}$:

$$\overline{BB} = \frac{9 + 11 + 10 + 15 + 13 + 11 + 10 + 11 + 12 + 11}{10} = 11.3 \times 10^{-3} \text{ in.}$$

2. The standard deviation, s, is calculated by the following equation:

$$s = \sqrt{(BB - \overline{BB})^2 / (n - 1)}$$

$$= \left[\frac{(9 - 11.3)^2 + (11 - 11.3)^2 + (10 - 11.3)^2 + \text{----} + (11 - 11.3)^2}{10 - 1}\right]^{1/2}$$

$$= 1.7 \times 10^{-3} \text{ in.}$$

3. The temperature correction is made to the average deflection according to Table 3. For a pavement temperature of 52 F, and an average deflection of $11.3 \times 10^{-3}$ in., the correction is $4 \times 10^{-3}$ in. Therefore,

$$\overline{BB}_{80} = \overline{BB} + 4 = 11.3 + 4 = 15.3 \times 10^{-3} \text{ in.}$$

4. The present design deflection is then calculated as

$$\overline{BB}_{80} + 2s = 15.3 + 2 \times 1.7 = 18.7 \times 10^{-3} \text{ in.}$$

5. The springtime deflection ratio for a 2.25 in. thick pavement tested March 24 is obtained from Table 6. Since this date is in the spring thaw period and beyond the table limit, a ratio of 1.0 is reasonably assigned. The design spring deflection, SBB, is obtained by multiplying the present design deflection from Step 4 by the springtime deflection ratio.

$$SBB = (\overline{BB}_{80} + 2s) \times 1.0 = 18.7 \times 1.0 = 18.7 \times 10^{-3} \text{ in.}$$

6. From Table 7 the allowable springtime deflection, ABB, is found for an ADT of 22 - 2700 (see Table 2) and a pavement thickness of 2.25 in.

$$ABB = 60 \times 10^{-3} \text{ in.}$$

7. The allowable springtime axle load, $L_A$, for this test section is estimated by the following equation.

$$L_A = L_D \frac{(ABB)}{(SBB)} = 9 \times \frac{60}{18.7} = 28.9 \text{ ton}$$

The mean $L_A$ is the average of the six $L_A$'s for Site No. 1a.

$$\text{Mean } L_A = \frac{28.9 + 28 + 31.6 + 32.3 + 22.9 + 21.3}{6} = 27.5 \text{ ton}$$

The overall mean $L_A$ is the average of the three mean $L_A$'s for Sites No. 1a., 2a., and 3a.

$$\text{Overall Mean } L_A = \frac{27.5 + 27.4 + 26.1}{3} = 27 \text{ ton}$$