Ordway Colorado Experimental Base Project

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The Ordway Colorado Experimental Base Project was a full-scale field experiment constructed with various thicknesses of two full-depth hot-mix sand asphalt bases, one full-depth asphalt concrete base, and one thickness of a standard design with untreated base and subbase layers and two different subgrades (A-6 and A-7-6). The project was planned as an American Association of State Highway Officials (AASHO) Road Test satellite project to be useful in extending the AASHO Road Test findings to Colorado conditions. The primary objectives were (a) to determine relative thicknesses of one asphalt concrete base, two hot-mix sand asphalt bases, and one standard design with untreated base and subbase required to give an equal level of pavement performance and (b) to relate certain measured properties of the pavement and the pavement components to observed levels of performance by using both empirical and theoretical models for pavement behavior. Performance data indicated that the different base types had different abilities to resist various forms of distress: (a) the asphalt concrete base mixture provided the best resistance to rutting and to all forms of cracking, (b) both hot-mix sand asphalt bases provided good resistance to alligator cracking but the low-stability hot-mix sand asphalt mix provided less resistance to rutting than the other bases, (c) the untreated standard base and subbase provided the best resistance to rutting but the least resistance to alligator or load-associated cracking. All base types exhibited severe transverse shrinkage cracking conditions. Analysis of rut depth and deflection data produced the following average layer coefficients: asphalt concrete surface course, 50 mm (2 in) thick, >0.44; asphalt concrete base course, 0.34; hot-mix sand asphalt base, 0.24-0.28; and untreated aggregate base and subbase, 0.16.

The Colorado Experimental Base Project was a full-scale field experiment constructed with various thicknesses of two full-depth hot-mix sand asphalt bases, one full-depth asphalt concrete base, and one thickness of a standard design with untreated base and subbase layers. The project was opened to traffic in 1965; routine measurements were discontinued in 1976. A final set of measurements was made in 1978.

The Colorado Experimental Base Project was planned as an American Association of State Highway Officials (AASHO) Road Test satellite project to be useful in extending the AASHO Road Test findings to Colorado conditions. The primary objectives were as follows:

1. To determine relative thicknesses of one asphalt concrete base, two hot-mix sand asphalt bases, and one standard design with untreated base and subbase required to give an equal level of pavement performance and
2. To relate certain measured properties of the pavement and the pavement components to observed levels of performance by using both empirical and theoretical models for pavement behavior.

The experiment was planned, designed, and constructed by the Colorado Department of Highways with the cooperation of the Federal Highway Administration and the Asphalt Institute. Continued surveillance and an extensive testing program were conducted by research personnel of the Department.

Several reports have been released to date on the project; they include summarized construction test data, field-performance measurements made during the first year, a subgrade moisture study, a deflection-temperature study, laboratory-determined mechanical properties of materials for elastic analysis, and a study of mechanical properties by using field dynamic-testing techniques (1-7). This report summarizes the field-performance data obtained during the period 1965-1978 and reports the results of analyses of the performance data. The final report (8) includes additional details of these analyses and tables of test data.

EXPERIMENT DESIGN

Two soil types, an A-7-6 with a California bearing ratio (CBR) of 2.6 and an A-6 with a CBR of 3.4, were included in the experiment, as shown in Table 1. Test sections were located within each soil type by a random process. There was a total of 20 test sections; each test section had a westbound and an eastbound lane. Test sections were 137.2 m (450 ft) long with an additional 15.2-m (50-ft) transition section at each end of the test section reserved for destructive testing.

Thicknesses were selected to give a maximum design life of approximately 20 years. Two thinner levels were selected in an attempt to obtain earlier failures. Traffic estimates made in 1978 indicated that the accumulated number of equivalent 80-kN (18 000-lbf) single-axle load applications (ESALs) applied between 1965 and 1978, when the last series of field measurements was made, totaled approximately 140 000.

MATERIALS AND CONSTRUCTION

The asphalt concrete surfacing had a well-graded aggregate and an asphalt content of 5.6 percent. The coarse aggregate contained at least 60 percent crushed material with at least two crushed faces. The asphalt concrete base contained slightly less fine aggregate than the surfacing but approximately the same percentage of asphalt. The aggregate used for the asphalt concrete base was similar to that used on the AASHO Road Test and was not crushed. The hot-mix sand asphalts contained 8 percent asphalt and 3-6 percent air voids. Average Marshall stability values for the sand asphalts were 2.0 kN (451 lbf) for the low-stability mix and 3.4 kN (770 lbf) for the high-stability mix. The higher stability of the high-stability mix was obtained by adding additional mineral filler to the low-stability mix. The Marshall stability of the asphalt concrete base was 5.2 kN (1170 lbf). The asphalt used for all three bases was a 60-70 penetration-grade asphalt with a mean penetration of 67.

The test sections identified as Colorado Standard were constructed by using an uncrushed gravel base and subbase. The base had a maximum size of 12.7 mm (0.5 in) and 9 percent passing the 75-μm (No. 200) sieve. The -425-μm (No. 40) sieve fractions were nonplastic. The subbase had a maximum size of 76.2 mm (3 in) and 5 percent passing the 75-μm sieve. It was nonplastic.

FIELD INSTRUMENTATION AND MEASUREMENTS PROGRAM

The objective of the field-measurement program was to determine the performance and load-response characteristics of the test section. Present serviceability index (PSI) was measured with the CHLOE profilometer developed on the AASHO Road Test (9,10). PSI was calculated by using the following formulas. The initial formula was as follows:

$$PSI = 5.03 - 1.91\log(SV - 2.0) - 0.01(C + P)^{1/2} - 1.38\,\overline{RD}^2 \quad (1)$$

where terms are defined as they were on the AASHO Road Test (9,10) and

Table 1. Experiment design.

	Soil Type				
	A-7-6			A-6	
	Base Thickness (in)				
Base Type[a]	140	178	216	140	178
Asphalt concrete (AC)	X	X	XX	X	X
Hot-mix sand asphalt					
Low stability (LSS)	X	X	XX	X	XX
High stability (HSS)	X	X	X		
Standard Design[b]		XXX			X

Note: 1 mm = 0.04 in.
X = one test section.

[a] Uniform 50-mm asphalt concrete surface course on all sections.
[b] 102-mm untreated base and 356-mm untreated subbase.

SV = slope variance from the CHLOE profilometer,
C + P = amount of class 2 and class 3 cracking plus patching (m^2/1000 m^2 of pavement), and
$\overline{RD}$ = average rut depth (mm/25.4).

Beginning in 1972, PSI was also corrected for texture by using the texturemeter described by Scrivner (10). A correction term was determined from tests on pavements that had a range in textures (11). A slight modification was also made in the PSI equation. The modified PSI equation used was as follows:

$$PSI = 4.85 - 1.91 \log(SV - 2.0) - 0.01(C + P)^{1/2} - 1.38\ \overline{RD}^2 \qquad (2)$$

The texture correction was made as follows:

$$PSI\ (corrected) = PSI + 0.140(T)^{0.560} \qquad (3)$$

where T is average texture determined with the texturemeter.

Rut depths were measured with the AASHO Road Test (9) rut-depth gauge for the routine-measurement program. The device measures the depth of rut over a 1.22-m (4-ft) span. Several randomly located spots were measured.

Cracking maps were prepared for each test section and updated periodically. Class 2 and class 3 cracks, identified as alligator cracks, were reported in square meters per 100 square meters of pavement. Class 2 cracks were defined (9,10) as cracking that has progressed to the stage at which the cracks become connected to form a grid-type pattern. Class 3 cracking was defined as that in which segments of the pavement surface have become loose. The number of transverse cracks and the total linear meters of all cracks, both longitudinal and transverse, also were reported.

The procedure adopted for the Benkelman-beam deflection measurement was the pavement-rebound procedure by using a 40-kN (9000-lbf) wheel load as published by the Canadian Good Roads Association (12) and the Asphalt Institute (13). For each test series, five locations in the outer wheel path of each lane of each test section were selected at random for testing. Thus, a total of 10 deflections was obtained for each test section.

Benkelman-beam deflection data were reported as recorded and as corrected to a standard temperature of 21.1°C (70°F). Temperature corrections were made by using curves developed in part from data collected on the project as described by Kingham (14) and which had been included in the Asphalt Institute publication (13) for many years. Temperatures were measured by using thermocouples until 1971, when the imbedded thermocouples were no longer giving satisfactory service. Afterwards the procedure described in the Asphalt Institute publication (13, Chap. XII) was used to estimate pavement temperatures. This procedure uses the pavement surface temperature and the preceding five-day mean air temperature.

Radius of curvature was determined by using the Dehlen curvature meter (15). Curvature data were not used in the analysis of performance data and are not reported in this paper.

LABORATORY MATERIALS CHARACTERIZATION STUDIES

The objective of the laboratory materials characterization program was to obtain data that could be used to apply elastic theory to a study of the pavement performance data. The tests included (a) triaxial compression resilient modulus M_r tests on the subgrade soils and untreated base and subbase materials, (b) unconfined dynamic modulus $|E^*|$ on the asphalt concrete surface and base mixtures and the two hot-mix sand asphalt base mixtures, and (c) repeated-load flexural fatigue tests on the asphalt concrete surface and base mixtures and the two hot-mix sand asphalt base mixtures. The tests were performed in the Asphalt Institute laboratory.

Repeated-load triaxial compression tests were performed to determine the resilient modulus M_r of the subgrade soils and the untreated granular subbase and base materials. Procedures and equipment for M_r determinations are described by Kallas and Riley (2) and in the Asphalt Institute Soils Manual (16). The data showed that the M_r of the subgrade soil--depending on soil type, water content, density, and the deviator stress and confining pressure test conditions--ranged between 27.6 and 227.5 MPa (4000 and 33 000 psi). The M_r of both the gravel aggregate base and subbase, depending on water content, density, and deviator stress and confining pressure test conditions, varied between 82.7 and 448 MPa (12 000 and 65 000 psi).

Dynamic modulus $|E^*|$ and phase lag ϕ were determined on laboratory-prepared samples of the asphalt concrete surface and base-course mixtures and on the low-stability and high-stability hot-mix sand asphalt base-course mixtures at temperatures of 4.4, 21.1, and 37.8°C (40, 70, and 100°F). Loading frequencies of 1, 4, and 16 Hz were used at each test temperature. Equipment and procedures for these tests have been described elsewhere (2).

Dynamic-modulus tests also were performed on cores to obtain information on modulus changes of the asphalt base courses with aging and traffic and for comparison with the modulus of laboratory-prepared specimens. Cores were obtained in 1965 shortly after the project was opened to traffic, in 1971 after the first pavement cracking was observed, and in 1976 at the time the final field measurements were made. Cores were obtained from wheel-path locations but not from the same test section at each sampling. Thus, modulus test results for cores include any variations due to the sampling location. Dynamic modulus $|E^*|$ and phase lag ϕ were determined at the same temperature and loading frequencies used for laboratory specimens except that the 1971 and 1976 core tests were also made with loading frequencies of 0.01 and 0.1 Hz at temperatures of -12.2 and 4.4°C (10 and 40°F) to obtain modulus data at lower temperatures and longer loading times.

Average values of $|E^*|$ for loading frequencies of 1, 4, and 16 Hz at each test temperature of 4.4, 21.1, and 37.8°C are shown in Figure 1. Average $|E^*|$ values for the asphalt concrete base show an increasing trend from 1965 to 1976. Average $|E^*|$ values for both low-stability and high-stability sand asphalt bases increased from 1965 to 1971 and then decreased from 1971 to 1976 to slightly below the 1965 levels. Average $|E^*|$ levels of asphalt concrete base were considerably above those of the sand asphalt base course.

Relationships between $|E^*|$ at 4 Hz and temperature for laboratory-prepared specimens and the 1965, 1971, and 1976 cores are shown in Figures 2 and 3, respectively, for the asphalt concrete base and the low-stability sand asphalt base. The $|E^*|$ of the laboratory specimens was about 12-33 percent greater than that for the 1965, 1971, and 1976 core specimens. Based on core test results, the modulus changes in all the asphalt base courses are relatively small and generally less than the difference between the modulus of laboratory-prepared specimens and the modulus of the cores.

Repeated-load flexure tests were made to determine constant stress fatigue behavior of the asphalt concrete surface and base courses and the high-stability and low-stability sand asphalt base courses. Equipment and procedures for the fatigue tests have been described by Kallas and Puzinauskas (17). Constants, correlation coefficients, and standard errors are given in Table 2 for the least-squares regression equation $N_f = k_1(1/\epsilon)^n$ for log-log plots of initial bending strain ϵ versus number of load applications to fracture N_f for the asphalt courses.

Test properties of the original asphalt and of asphalt extracted from the 1976 asphalt base and surface course cores are given in Table 3. Compared with the test properties of the original asphalt, there was little change in the consistency of the asphalt in the asphalt base courses from 1965 to 1976. The asphalt hardened slightly more in the asphalt concrete base than in the sand asphalt base courses. The asphalt hardened considerably more in the asphalt concrete surface course than in the base course.

ANALYSIS OF ROUTINE FIELD-PERFORMANCE DATA

An overall picture of the relative performance of different test sections at the end of the measurement period can be found in Table 4, a summary of measurements made in September 1978. The 1978 data were the last set obtained on the project. Measurements included in Table 4 are PSI, rut depth, cracking, and deflection. Averages for both eastbound and westbound roadways are given.

In 1976 and 1977, a preliminary study was made of selected data collected between 1970 and 1976 by using analysis-of-variance (ANOVA) techniques. The following measurements were included in the study: PSI, corrected for texture; rut depth; linear meters of cracking per 1000 square meters of pavement; alligator cracking; number of transverse cracks; linear meters of transverse cracking; and Benkelman-beam deflection, corrected for temperature. The purpose of the analysis was to determine base type or base thickness effects as measured by these parameters, one of the major objectives of the project. The analysis was made for the test sections on the A-7-6 soil type (Sections 1 through 14) and for each year and each lane, where data permitted. A conventional two-way ANOVA with a single observation per cell and partial replication was used (18).

Figure 1. Average dynamic modulus of cores.

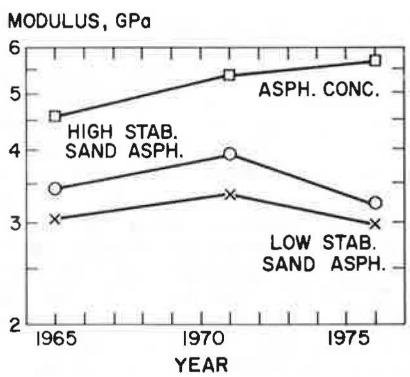


Figure 2. Temperature versus dynamic modulus: asphalt concrete base.

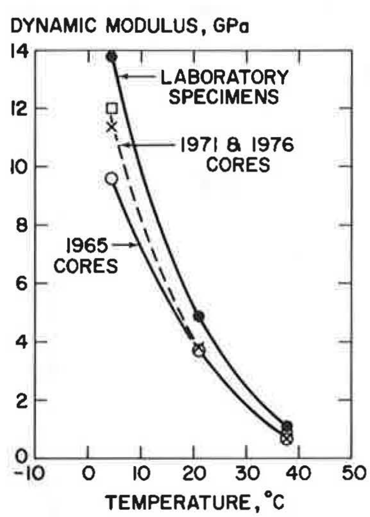


Figure 3. Temperature versus dynamic modulus: low-stability sand asphalt base.

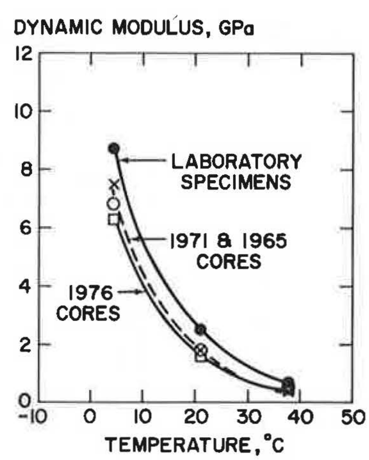


Table 2. Results of flexural fatigue tests on asphalt mixtures as function of strain.

Course	No. of Specimens	Temperature (°C)	Constant k_1	Constant n_1	Correlation Coefficient	SDE
AC surface	9	21	2.73×10^{-7}	3.25	0.91	0.52
AC base	8	21	2.01×10^{-5}	2.69	0.98	0.18
LSS asphalt	3	21	8.97×10^{-7}	3.25	0.99	0.13
HSS asphalt base	2	21	2.82×10^{-11}	4.60	-	-

Note: $t°C = (t°F - 32)/1.8$.

Results of the ANOVA indicated that, in general, there were no highly significant differences between base types or base thicknesses for most of the measurements and years included in the analysis. No significant differences in PSI could be attributed to either base type or base thickness. Visual inspection of the 1978 data summarized in Table 4 confirms this observation. A regression analysis of the PSI trend data also confirmed the conclusion, although there was a slight effect of thickness in this study.

The fact that there is almost no effect of base thickness on PSI loss indicates that most of the loss in PSI observed after 12 years of traffic was not load associated. The absence of alligator cracking in the sections with thin treated bases also supports this thesis. Surface erosion of the matrix from the surface of the pavement had a large effect on the PSI measurements made with the CHLOE profilometer and indicates that most of the PSI loss can be laid to environmental influences. The presence of alligator cracking in the standard sections with thin asphalt surfacing and untreated bases and the presence of a small amount of rutting in all sections indicates that axle loads did have an effect on the pavements, however.

A considerable amount of transverse cracking, possibly a low-temperature cracking phenomenon, was observed on the project. However, there does not appear to be any relationship between this cracking and PSI. Transverse cracking was first observed on the project in 1971 after a particularly severe winter. Some longitudinal cracking was observed prior to this in untreated base section 20 and in the paved shoulders, but it was not extensive.

Three major types of cracking were logged in a study of the cracking maps: alligator cracking, transverse linear cracking, and longitudinal linear cracking. The distribution of the three types of cracking is summarized below:

Base	Percentage of Cracking by Type (1978)		
Type	Alligator	Transverse	Longitudinal
AC	2	70	28
LSS	4	70	26
HSS	3	73	24
Standard	30	38	32

Percentage of cracking is defined in this study as the percentage of blocks, or arbitrary units of area, 0.61 m (2 ft) long (direction of traffic) and 0.3 m (1 ft) wide (transverse direction) that exhibit cracking of the type indicated.

It is apparent that alligator cracking distress was a major factor only in the untreated base test section. In 1978 these sections averaged 9 percent alligator cracking after 140 000 load repetitions. Alligator cracking is usually associated with pavements that exhibit fatigue distress.

By far the most prevalent form of cracking in the asphalt base sections was transverse and longitudinal linear cracking. The extent of this cracking can be observed in Figure 4. The first analysis of this type of cracking was made by Kingham in 1972 shortly after it was observed. Kingham observed that the transverse orientation of the cracks ob-

Table 3. Test properties of original asphalt and asphalt recovered from 1976 base and surface-course cores.

		Asphalt Extracted from 1976 Cores			
Test Property	Original	AC Base	HSS Base	LSS Base	AC Surface
Viscosity					
15.5°C (Mpoises)	26				
60°C (poises)	2546	3499	3147	3001	11 023
135°C (cSt)	427	545	478	459	800
Penetration, 100 g, 5 s, 0.1 mm	60	55	55	60	31
Softening point (ring and ball) (°C)		53.9	53.3	52.8	59.4
Ductility, 25°C (5 cm/min) (cm)	150+				
Specific gravity, 25°C	1032				
Thin-film oven residue:					
Penetration, 25°C (100 g, 5 s)	40				
Viscosity, 60°C (poises)	5636				
Viscosity, 135°C (cSt)	627				

Notes: $t°C = (t°F - 32)/1.8$; 1 mm = 0.04 in; 1 cm = 0.39 in.

Table 4. Summary of measurements made September 1978.

Section No.	Base Type	Pavement Thickness (mm): Asphalt Surface and Base	Pavement Thickness (mm): Untreated Base and Subbase	PSI	PSI Corrected[a]	Rut Depth (mm)	Percentage of Cracking[b]: Alligator	Percentage of Cracking[b]: Total	Deflection[c] (mm)
7	AC	201		1.6	2.5	5	1	22	0.36
12	AC	234		1.7	2.7	5	1	26	0.25
4	AC	279		1.8	2.6	3	1	18	0.15
8	AC	269		1.6	2.4	5	1	20	0.30
16	AC	190		1.9	3.0	5	0	7	0.40
15	AC	254		1.8	2.5	5	1	8	0.30
17	AC	236		1.6	2.5	5	0	7	0.30
10	LSS	196		1.6	2.5	10	1	10	0.46
2	LSS	239		1.8	2.6	10	1	12	0.33
3	LSS	297		2.0	2.6	10	1	16	0.25
13	LSS	269		1.6	2.5	8	1	15	0.30
18	LSS	193		2.0	2.4	13	1	15	0.38
19	LSS	234		2.0	2.3	10	1	7	0.36
5	HSS	198		1.6	2.5	8	1	25	0.41
9	HSS	226		1.7	2.7	5	1	19	0.33
6	HSS	274		1.7	2.4	8	1	14	0.30
1	Standard	71	404	1.7	2.7	3	2	25	0.41
11	Standard	66	442	1.9	2.4	3	12	35	0.30
14	Standard	56	422	1.8	2.5	3	16	36	0.33
20	Standard	71	409	1.9	2.6	3	7	22	0.25

Note: 1 mm = 0.04 in; $t°C = (t°F - 32)/1.8$.

[a] Corrected for texture.
[b] Percent total area cracked.
[c] Benkelman-beam deflection taken after spring thaw, corrected to 21.1°C.

Figure 4. Transverse shrinkage cracking.

served after the severe winter pointed to the likelihood that they resulted from thermal shrinkage, a phenomenon that at that time was attracting considerable attention in Canada and in parts of the United States. Kingham used techniques in his investigation subsequently reported by Haas (19). He used the results of original tests on the asphalt from the project and data obtained from asphalt concrete core samples obtained from the roadway. Critical temperatures when cracking first appeared were of the order of -21.7°C (-7°F) for the asphalt pavement. A -30°C (-22°F) air temperature was observed. By using properties of the asphalt cement and the core samples, Kingham was able to conclude that the pavements would be expected to crack at these temperatures. He stated, however, that his investigation was limited and that further investigations should be made.

In subsequent years, the amount of linear cracking increased, even though winter temperatures never were as low as those observed at the time the cracking was initiated. In general, the greatest amount of cracking in test sections constructed with hot-mix sand asphalt bases occurred in 1971, whereas the greatest amount occurred in the asphalt concrete base and standard sections in 1974 or in later years. These differences were not great, however.

Rut-depth data from Table 4 for the asphalt base sections of approximately equal thickness are summarized below for comparison (1 mm = 0.004 in; range is distance between sections):

Base Type	Location	Rut Depth (mm) Avg	Range	Approximate Base Thickness (mm)
AC	West	5	0	165
LSS	West	10	0	165
HSS	West	8	3	165
AC	East	5	0	165
LSS	East	13	3	165

The summary indicates that, when compared on the basis of equal thickness, the hot-mix sand asphalt bases deformed more than the asphalt concrete base did and that the high-stability hot-mix sand asphalt mix was more resistant to deformation than was the low-stability mix. The rut depths observed for the standard untreated base sections were less than any observed for the asphalt-treated base sections, except Section 4 (279 mm of asphalt concrete).

An attempt was made to investigate the relative resistance to rutting of the different bases through the application of linear regression techniques. Two analyses were made: a comparison between the asphalt-treated bases and a comparison between the asphalt concrete base and the standard base. Unfortunately, the analysis between the asphalt bases was not successful because of the large amount of scatter in the data. However, the results did confirm the fact that the different bases performed differently, and they were useful in calculating the layer equivalency values reported later.

Routine Benkelman-beam deflection data, taken in the spring after thaw, and radius-of-curvature data are summarized in Table 4 for 1978. A simple linear regression analysis and 1978 data were used to determine whether there was a significant effect of base thickness or base type on deflection. The following model was used in analysis:

$$\text{deflection} = a_0 + a_1 T + a_2 C \quad (4)$$

where

a_0, a_1, a_2 = regression coefficients,
T = base thickness,
C = base code = 0 for asphalt concrete, and
C = base code = 1 for LSS and HSS bases.

The 1978 data points and a plot of the derived curves can be seen in Figure 5.

In this analysis both the base-thickness term and the base-type code term were statistically significant from zero; i.e., deflection reflected both base type and base thickness. Comparisons on the basis of equal deflection were made from the equations. Approximate ratios of 1.1 to 1.2 were obtained for thickness of low-stability and high-stability sand asphalt compared with asphalt concrete. A similar comparison was made between the average thickness of standard untreated base and the calculated thickness of asphalt concrete for the average deflection of the untreated base. The deflection value in this case was 0.33 mm (0.013 in) and the thickness ratio was 1.9 (419/216).

SPECIAL STUDIES

Several special field studies, not considered part of the routine measurement program, were made during the life of the project: These included a continuing study of subgrade moisture conditions, special deflection studies, and a trench study.

Subgrade moisture studies were made in a number of test sections in 1965, 1966, 1967, 1968, 1969, 1971, 1972, and 1974. Figure 6 shows schematically the distribution of moisture content obtained in 1968 test sections 3 and 7. Table 5 summarizes moisture data obtained from sections 3, 7, and 11 on a routine basis in the years 1967, 1968, 1969, 1971, 1972, and 1974 for the top 305 mm (1 ft) and second 305 mm on the centerline and approximately 3.05 m (10 ft) on either side of the pavement centerline. From earlier studies of data such as those presented in Figure 6, it appeared that the subgrade under the full-depth asphalt sections was drying out, whereas under the untreated base sections the subgrade was getting wetter. This appears to be the case from Figure 6, but it is not clearly evident from the data in Table 5.

Large seasonal variations in deflection were observed in the full-depth asphalt sections, which appeared to be related to differences in temperature rather than moisture or frost effects. A study of temperature effects was initiated in 1966. The study was designed to measure pavement deflections over a temperature range of -1°C to 37.8°C (30-100°F). Two levels of subgrade strength, temperature, and thickness were included. Low subgrade strength conditions were evaluated in the spring and high strength conditions were evaluated in the late summer or fall. The program was completed over a two-year time period, 1966-1968. These data along with similar data collected on the San Diego County Experimental Base Project (20) were used to develop temperature-deflection correction curves (13).

In late 1976, trenches were cut in the eastbound lanes of Sections 10 and 18 to study more closely the rutting that had occurred in these sections. Sections 10 and 18 consisted of a nominal 140-mm (5.5-in) thickness of low-stability hot-mix sand asphalt base. Rut-depth data from the trench study indicated that the change in thickness of the surface course was approximately 5 percent, or about 13 percent of the total rut depth. The hot-mix sand asphalt base was reduced in thickness about 10 percent or about 73 percent of the rut depth. About 14 percent of the rut was attributed to the subgrade.

Since the above study involved only one base type and one base thickness, no comparisons can be made with other base types or thicknesses. However, the data do confirm that the source of the rut is primarily the base and that from a structural viewpoint the subgrade was not significantly overstressed.

Figure 5. Deflection versus base thickness.

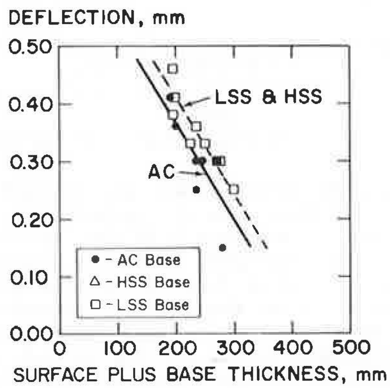
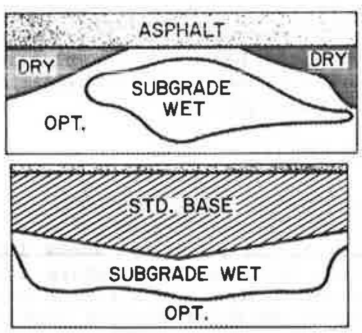


Figure 6. Results of moisture study.



RELATIVE THICKNESS REQUIREMENT OF PAVEMENT MATERIALS

In the American Association of State Highway and Transportation Officials (AASHTO) Interim Guide (21) and the Colorado Roadway Design Manual (22), the relative contributions of different types of base and subbase are expressed as coefficients in an equation of the form shown below:

$$SN = aD_1 + bD_2 + cD_3 \tag{5}$$

where

- SN = structural number, obtained from the design method for a given design situation;
- a, b, c = coefficients that vary with material type; and
- D_1, D_2, D_3 = thickness of surface, base, and subbase layers, respectively.

A list of coefficients used in the AASHTO and Colorado design methods is given in Table 6.

Two of the analyses presented in this report, the rut-depth analysis and the deflection analysis, produced results that were used to estimate material coefficients for the materials used as base on the project. These values are given in Table 7.

In addition to the coefficients for the base materials, there are indications from the analysis of rut depth presented earlier and from an analysis of deflection data not included that the coefficient for the asphalt surface course should be higher than 0.44. The values obtained were 0.94 and 0.48, respectively. It would seem, however, that the frequently used value of 0.44 would be more reasonable for use in routine design.

Table 5. Summary of routine subgrade moisture measurements.

		Percentage of Moisture									
		Section 3			Section 7			Section 11			
Year	Subgrade Depth[a] (mm)	N[b]	C	S[c]	N	C	S	N	C	S	Avg All Sections
1967	Top 0.3 m	15	21	15	11	14	15	20	20	20	17
	Next 0.6 m	16	20	14	14	17	17	18	18	18	17
1968	Top 0.3 m	18	21	15	11	10	15	7	10	9	13
	Next 0.6 m	17	21	19	14	18	17	19	21	21	19
1969	Top 0.3 m	13	21	20	19	19	18	19	19	18	18
	Next 0.6 m	15	21	20	20	19	20	22	22	22	20
1971	Top 0.3 m	15	19	18	16	16	16	16	20	17	17
	Next 0.6 m	20	22	21	20	20	21	23	24	23	22
1972	Top 0.3 m	13	11	13	15	16	13	14	15	17	14
	Next 0.6 m	19	21	21	20	21	19	22	22	22	21
1974	Top 0.3 m	20	19	20	19	9	18	19	20	19	18
	Next 0.6 m	20	20	19	21	14	20	24	23	23	20
Avg	Top 0.3 m	16	19	17	15	14	16	16	17	17	16
	Next 0.6 m	18	21	20	18	18	19	21	22	22	20
Section avg	Top 0.3 m		17			15			17		
	Next 0.6 m		20			18			22		
	Top 0.9 m		18			16			19		

Note: 1 mm = 0.04 in.
[a]Approximate depth. [b]N = approximately 3 m north of centerline of roadway. [c]S = approximately 3 m south of centerline of roadway.

Table 6. Comparison of layer coefficients.

Material	AASHTO Guide Coefficients[a]	Colorado Method Coefficients[b]
AC surfacing (D_1)	0.44	0.25-0.44
AC base (D_2)	0.34	0.22-0.34
LSS base (D_2)	0.30	Not included
HSS base (D_2)	0.30	Not included
Standard base (D_2)	0.14[c]	0.10-0.14
Standard subbase (D_3)	0.11	0.10-0.14

[a]Adopted in 1961; values used by many individual states are different.
[b]Functions of strength of material as measured by R_t-values for asphalt mixes and R-values for aggregate bases.
[c]Normally applied to crushed stone.

Table 7. Estimated layer coefficients.

Base Material	Rut-Depth Analysis		Deflection Analysis		Average	
	Ratio	Coefficient	Ratio	Coefficient	Ratio	Coefficient
AC	1.0	0.34	1.0	0.34	1.0	0.34
LSS	1.8	0.19	1.2	0.28	1.5	0.24
HSS	1.2	0.28	1.2	0.28	1.2	0.28
Standard[a]	2.4	0.14	1.9	0.18	2.2	0.16

[a]Includes base and subbase.

RESISTANCE TO RUTTING AND CRACKING

Based on information reported in Table 4, it is possible to rank the base types in order of their resistance to a given form of distress or performance variable; i.e., roughness (PSI), rut depth, alligator cracking, and transverse shrinkage cracking. The results of this ranking process are given below (0 = equal, 1 = best, etc.):

Base Type	Rank by Performance Variable: Roughness (PSI)	Rut Depth	Alligator Cracking	Transverse Cracking
AC	0	2	0	1
HSS	0	3	0	4
LSS	0	4	0	3
Standard	0	1	4	2

CONCLUSIONS

The different base types included in the Ordway Colorado Experimental Base Project exhibited different abilities to resist various forms of distress:

1. The asphalt concrete base mixture provided the best resistance to rutting and to all forms of cracking,
2. Both hot-mix sand asphalt bases provided equally good resistance to alligator cracking, but the low-stability hot-mix sand asphalt mix provided less resistance to rutting than the other bases, and
3. The untreated base and subbase provided the best resistance to rutting but the least resistance to alligator or load-associated cracking.

After 12 years of traffic without major maintenance, all test sections were exhibiting substantial erosion of the matrix from the asphalt concrete surface course and severe transverse shrinkage cracking conditions. These conditions were somewhat more severe than was observed on pavements on adjacent highways. Also, some of these conditions may have been partly overcome by normal maintenance procedures; however, because of the experimental nature of the project, no maintenance was permitted.

Analysis of rut-depth and deflection data produced the following average layer coefficients: asphalt concrete surface course, 50 mm (2 in) thick, >0.44; asphalt concrete base course, 0.34; hot-mix sand asphalt base, 0.24-0.28; and untreated aggregate base and subbase, 0.16.

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The contents of this report reflect our views and we are responsible for the facts and accuracy of the data presented here. The contents do not necessarily reflect the official views of the Asphalt Institute, the Colorado Department of Highways, or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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Use of Deflection Measurements for Determining Pavement Material Properties

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This paper develops and describes models and constraints for using Dynaflect measurements to obtain the elastic-modulus inputs for layered theory. A nomograph is provided for determining the subgrade modulus of elasticity by using the sensor-1 and sensor-5 deflections. A graph and equations for correcting these modulus properties based on the thickness of the subgrade are also provided. In addition, problems associated with the modulus predictions considering stress sensitivity of pavement materials, variations of subgrade stiffness with depth, seasonal effects, and discontinuities in the pavement structure are described. A step-by-step summary procedure is provided to permit a designer to readily utilize the information presented in the body of the report.

Mechanistic design procedures require the use of a suitable theory and model to analyze the behavior of a pavement structure. Plate, layered, and finite-element theories have been used for this purpose. Typically, these theories are used to compute the tensile stresses in the upper, bound pavement layers, which are then input into a fatigue equation to predict the life of the pavement. Use of one of these theories requires that the materials that make up the pavement be suitably characterized.

Plate theory is often used for rigid pavement design; if so, the concrete layer is represented by a relatively stiff plate and the lower layers are characterized as a bed of linear springs. Elastic-layered and finite-element theories have also been used with success for rigid pavement design. These last two theories use Young's modulus and Poisson's ratio to characterize the stress-strain behavior of the pavement materials.

Taute, McCullough, and Hudson (1) have shown that plate and layered theories predict similar tensile stresses in the bottom of a concrete pavement layer when the supporting structure consists of granular material. The spring constant K used in the plate-theory calculations is equated to the layer moduli used in layered-theory calculations by computing the deflection of the subbase under a plate load with layered theory. This deflection is used to obtain the equivalent k-value of the supporting structure.

Layered-theory computer programs that can predict the state of stress, strain, and deflections of pavement structures at minimal cost are freely available. For this reason, layered theory is often used in mechanistic design procedures. Shortcomings of the theory, such as the inability to predict pavement stresses under an edge-loading condition, can be overcome by using stress-modification factors. These factors can be calculated by using plate or finite-element theories.

OBJECTIVE

Because layered-theory analysis is often used for mechanistic pavement analysis, the material properties most often required for the pavement layers are Young's modulus and Poisson's ratio. Both laboratory and in situ methods are available for determining these material characteristics. The objective of this paper is to develop and describe the techniques, models, and constraints involved in using deflection measurements to obtain the inputs to layered theory.

DEFLECTION MEASUREMENTS

The use of deflection measurements for the estimation of pavement layer stiffnesses is rapidly gaining popularity and application. Computer programs that model the pavement layers as homogeneous, isotropic, elastic layers provide reasonable estimates of pavement behavior under loading. A Dynaflect is at present used in Texas to obtain pavement deflection measurements. Thus, the developments in this paper are based on Dynaflect loadings, but the concepts are applicable to any deflection-measuring device.

The Dynaflect uses two masses rotating in opposite directions to apply a cyclic load to the pavement surface. The cycle frequency used is typically 8 Hz and the peak-to-peak load applied is 1000 lb on two steel load wheels placed at 20-in centers. The peak-to-peak deflections are measured by five geophones at 1-ft intervals; the first one is placed