Incremental Design of Flexible Pavement

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Deficiencies and problems inherent in existing empirical and mechanistic design procedures have prompted an incremental design approach based primarily on the response of in-situ pavement to nondestructive tests. The first increment is designed with a structural section sufficient to preclude excessive subgrade deformation using conventional pavement design procedures. The second, if necessary, is placed in sufficient thickness to avoid excessive tensile strain of the asphalt concrete surfacing and, thus, premature fatigue cracking. A field evaluation was conducted on incremental sections with and without fabric reinforcement, and on control sections designed using the standard California (empirical) procedure. On the basis of laboratory tests characterizing all elements of the structural section, pavement designs for the site were developed using two empirical and one mechanistic-empirical procedure. These designs were compared to that designed by the incremental procedure. The results revealed that substantial savings in initial structural section costs are possible by incremental pavement design.

As defined by C. K. Kennedy, "Pavement design is essentially a structural engineering problem in which it is necessary to define the load conditions and to ensure that, for the range of environmental conditions to which the pavement is subjected, the materials can absorb the stresses and strains imposed without suffering unacceptable deterioration" (*1*). More specific to the semiarid environment representative of most of California, flexible pavement design has, as its primary objectives, avoidance of two conditions that will result in premature distress. As shown by Figure 1, one of these is permanent subgrade deformation, ultimately resulting in pavement rutting. The second is excessive tensile strain, which induces fatigue cracking within the design life of an asphalt concrete (AC) surfacing.

EMPIRICAL DESIGN

Empirical flexible pavement design procedures, including the California (*R*-value) and AASHTO methods, are primarily aimed at avoiding excessive subgrade deformation. All are based on standard soil strength tests, accelerated test track data, and/or long-term performance of in-service pavements.

California procedure is based, in part, on a triaxial-type soil strength test (*R*-value) and the results of the Brighton test track in California conducted in the early 1940s. In 1957, the traffic factor was modified so that the California procedure would conform to the results of the WASHO Road Test (*2*). As a result, structural section thickness increased an average of 3 in.

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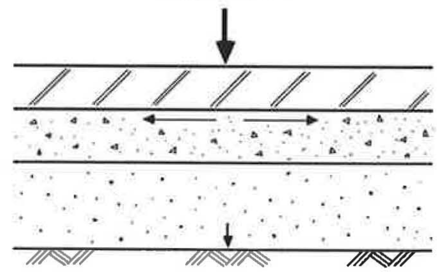

FIGURE 1 Subgrade deformation and AC tensile strain criteria used for flexible pavement design.

In 1959, the procedure was adjusted further based on the assumption of a greater degree of base, subbase, and subgrade saturation than was previously utilized (*3*). This modification resulted in a significant decrease in *R*-value, particularly on subgrade soils with a high silt content. In 1963, additional adjustments were implemented using the results of the AASHO Road Test (*4*). The *R*-value, and gravel equivalence for asphalt concrete, and the method of calculating the traffic factor for the design equation were all modified so that required pavement thickness was increased. As a result of these modifications, evidence of permanent subgrade deformation (rutting) is extremely rare in California, even on badly cracked asphalt concrete pavements.

In 1982, a further modification of the flexible pavement design policy in California was made to ensure rapid positive drainage of water entering the pavement surface. The Caltrans design manual was modified in 1987 (*5*), based upon the experience of the previous four years, to simplify the procedure. Except in areas of extremely low rainfall (<5 in./yr), or where a relatively permeable subgrade is available (≥100 ft/day), new structural section designs incorporate a drainage element with collectors and outlets. Rapid drainage is normally accomplished by an asphalt-treated permeable base with a permeability of ± 15,000 ft/day. Positive rapid drainage of the pavement structural section should extend pavement life significantly and/or permit reductions in pavement thickness, the extent of which has not, as yet, been determined. The quantification of the benefits of positive rapid pavement structural section drainage should be given a high priority in future flexible pavement research.

The empirical pavement design procedure utilized by California, in common with those used by other entities with which the writers are familiar, share the following deficiencies:

1. They are usually based on a worst-case subgrade strength

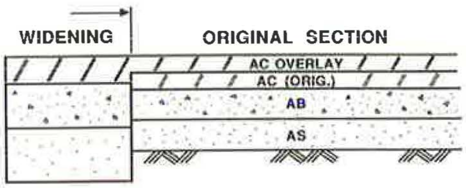

FIGURE 2 Overlay design for a thin structural section (AC = asphalt concrete, AB = aggregate base, and AS = aggregate subbase).

condition both in terms of material properties and degree of saturation.

2. They have been developed from accelerated test track data which do not adequately incorporate the critically important performance variable of asphalt concrete—resistance to fatigue cracking.
3. They do not allow for the effects of positive rapid pavement drainage.
4. They are based on a relatively limited range of traffic loading conditions.

The innate conservatism of California's present empirical design procedure has also been demonstrated repeatedly upon application of the overlay design procedure in connection with widenings. As shown by Figure 2, almost without exception, overlay design requirements of an existing relatively thin structural section, as determined by nondestructive tests (pavement deflection measurements), are such that the existing structural section, plus required overlay, is usually substantially thinner than that required by the conventional *R*-value design procedure. This phenomenon was graphically illustrated by an investigation in March 1980 by the Transportation Laboratory (*6*). That study required recommending a structural section design for a widened portion of Bottle Rock Road in Lake County and an overlay design for the existing pavement. In the past, that road had performed remarkably well in spite of its thin structural section (in some cases, as little as 0.1 ft of asphalt concrete over existing soil). On the basis of conventional design procedure, the following structural section was found to be necessary for the widened portion of Bottle Rock Road: 0.4 ft asphalt concrete, 0.5 ft aggregate base, and 1.15 ft aggregate subbase. On the basis of the results of the nondestructive test conducted on existing structural section, the design of the widened portion was modified to 0.35 ft of asphalt concrete over 0.5 ft of aggregate base. The use of nondestructive tests on the existing facility as the basis of the widening design resulted in a 43 percent savings in structural section construction costs.

MECHANISTIC DESIGN

In recent years there has been a concerted effort on the part of many in the pavement design community to move from empirical to mechanistic design, a system by which elements of the structural section are modeled by elastic-layer theory or by finite-element analysis. Design criteria require sufficient layer thickness to ensure that asphalt concrete tensile strain and subgrade deformation do not exceed tolerable limits.

Practical application of mechanistic procedures to routine pavement design presents real difficulties including the use of simplifying theoretical assumptions and material characterization procedures. Thus, it has been necessary to calibrate mechanistic designs with observed pavement behavior, utilizing "shift factors." Other problems associated with mechanistic design are appropriate characterization of typically stress-dependent and moisture-sensitive subgrade materials, extent and effects of seasonal changes in the moisture regime, and changing properties of the asphalt concrete due to aging.

For current use, and in the near future, mechanistic procedures show more promise in analysis than in routine design. Mechanistic procedures have already been used by Caltrans and others as an analytical tool to evaluate new materials, unique structural sections, and loading conditions.

Although mechanistic design has a much sounder basis in structural theory than does conventional empirical design, its reliance on uncertain pavement loading and materials properties data makes it, at best, a *mechanistic-empirical* procedure. Future research may eventually overcome the problems in the mechanistic approach, but this does not provide today's pavement design engineer with a cost-effective and efficient design tool for routine use.

INCREMENTAL DESIGN

Another approach to flexible pavement design which is currently being evaluated by Caltrans has been identified as "incremental design." This approach, not to be confused with stage construction, requires design of the pavement in two separate increments. The first addresses excessive subgrade deformation. It can be developed by the use of a mechanistic procedure or an empirical design based on a 3- to 5-year traffic loading.

The second increment is developed during construction on the basis of nondestructive tests conducted on the first increment. From the resulting data, an overlay of sufficient thickness to preclude premature fatigue cracking completes the pavement. The basis of the design of the second increment is the Caltrans Overlay Design Procedure, the development of which began in 1938 with the installation of permanent linear variable differential transformer gages in pavements throughout California. In 1955, critical levels of pavement deflection were determined for average traffic loading conditions, which, if exceeded, would result in premature fatigue cracking (*7*). An overlay design procedure based on these critical deflection levels was adopted in 1965 along with a means of varying critical levels for traffic conditions. The critical deflection–ESAL–thickness curves were adjusted slightly as a result of a 14-year study of 27 test pavements completed in 1978 (*8*).

In 1980, further follow-up studies (*9*) on 40 selected pavements designed by the overlay procedure indicated an average life of 11 years (see Figure 3). The overlay design procedure has stood the test of time and is deemed an appropriate and reasonable procedure to develop designs sufficient to preclude fatigue cracking of an asphalt concrete pavement under average conditions in California.

The basic rationale for incremental pavement design was aptly stated by C. R. Foster in his discussion of the AASHO

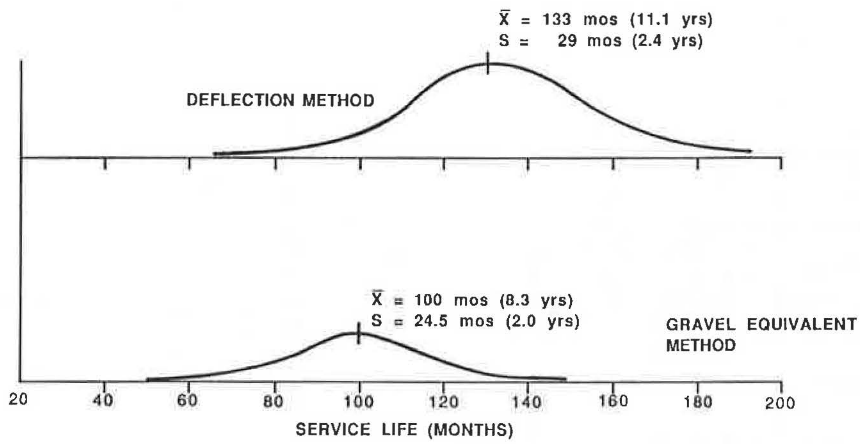

FIGURE 3 Service life of structural overlays.

Road Test at the 1962 International Conference on the Structural Design of Pavements (*10*):

> [P]erformance of a pavement under traffic is related only to the in-place strength of the materials in the pavement cross section. It is not related to the strengths predicted by a laboratory design test (although the laboratory test for predicting strength is a necessary part of any design system), it is not related to the as-constructed strengths; nor is it related to the strengths measured in loop 1 which received no traffic. Performance is related only to the in-place strengths in the section being trafficked.

Put another way, using proven nondestructive testing techniques, incremental design offers the possibility of utilizing the most cost-effective and efficient design tool available, the response of the prototype to load.

The potential economics in an incremental approach to flexible pavement design suggested by over 20 years of pavement deflection studies in California prompted the development of the field evaluation that is reported herein. An integral part of that evaluation also involves a comparison of the incremental-design section with designs developed for the same site by conventional empirical and mechanistic procedures.

Test Project Description

The first project selected for field evaluation of the incremental procedure is on a section of Route 99 between Sacramento and Marysville/Yuba City. This facility was recently widened to provide additional capacity and safety for significant increases in truck and automobile traffic. The previous alignment consisted of two-lane roadway traversing farmland that is subject to rice farming and irrigation. The existing basement soils in the area are silty clays and claying silts of low strength when saturated.

This project consisted of widening the existing roadway and constructing a separate two-lane southbound roadbed. The existing two-lane roadway now serves as the northbound lanes. The final grade is 3 to 4 ft above adjacent irrigated farmland.

Construction began during the spring of 1986. The limits of this contract were from 0.5 mile north of Route 5 to 0.8 mile north of Elverta Road, about 10 miles north of Sacramento.

Experiment Design

Because of the generally uniform, and relatively weak, subgrade conditions (R-value = 5) the site also provided an excellent opportunity to evaluate the use of high-strength geotextiles for subgrade enhancement.

The experimental and control structural sections were developed based on the California (R-value) design procedure and are shown in Figure 4. In addition to the incremental design, a high-strength woven fabric for subgrade enhancement was also included. The test section from Stations 175 to 180 is designed with an "effective" R value of 40 based on subgrade support provided by the fabric, as shown by recent research of the Transportation Laboratory (*11, 12*). A second test section from Station 180 to 185 is basically the incremental design underlain with the same fabric. The test sections between Stations 170 and 175 and between 195 and 200 are control sections designed on the basis of 5 R-value basement soil and a traffic index (TI) of 12.5 over a 20-year design period. The incremental sections are designed for the same basement soil conditions and a TI of 10.0 for a 5-year design period.

The conventional section for the remainder of the project was also designed for 20 years and was placed over imported material with minimum R-value of 15. The upper 1.0 ft of the imported material was treated with lime. The field experiment was extended from Station 200 to 210 to include a 1,000-ft test section (see Figure 4).

During the field-testing period, it was proposed that the constructability and performance of the various experimental test sections be evaluated by materials sampling and by the following tests:

1. Sampling basement soil, aggregate subbase (AS), aggregate base (AB), asphalt-treated permeable base (ATPB), and AC surfacing and performing applicable laboratory tests for grading, R-value, sand equivalent (SE), density, stability, and resilient modulus (M_r).
2. Obtaining samples of proposed geotextile fabric and testing for grab tensile strength and elongation.
3. Obtaining deflection measurements with the Dynaflect during construction, on the subgrade and on each subsequent layer of structural section.

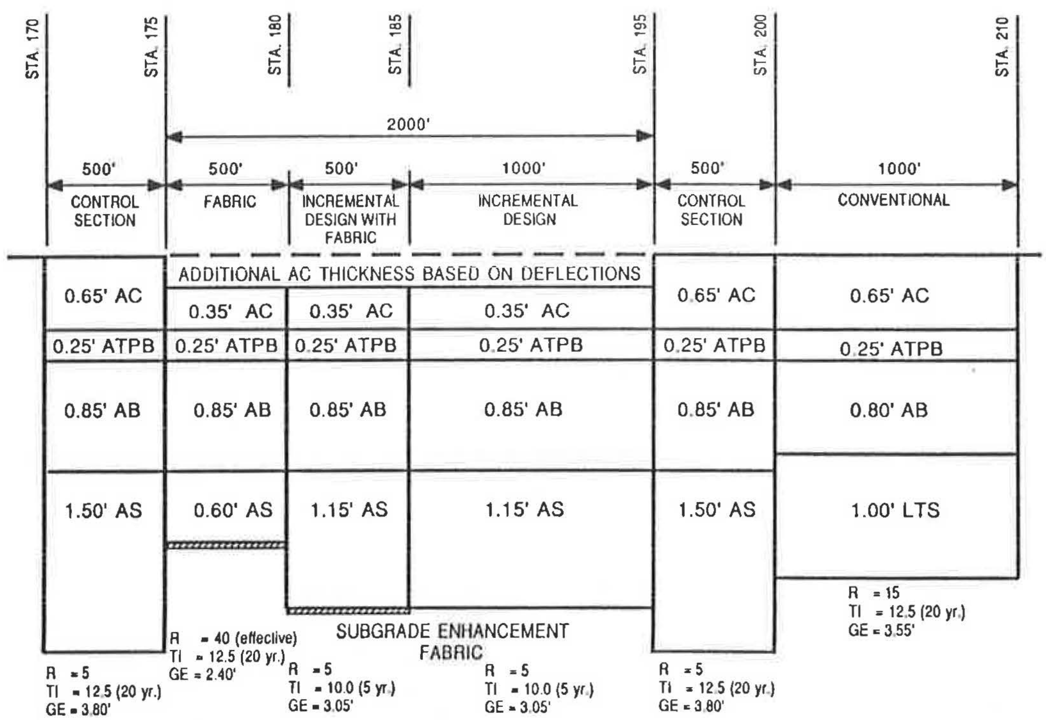

FIGURE 4 Experimental test section: 3-Sac-99-P.M.32.6/36.2.

4. Observing structural section construction progress and noting any difficulties or advantages of using subgrade enhancement fabrics.
5. Using deflection measurements on AC surface course to determine need for additional AC to satisfy structural requirements of the structural section for each of the proposed test sections.

Ultimately, the performance of the incremental sections would be evaluated in terms of what would have been required under test site subgrade and loading conditions using conventional empirical and mechanistic design procedures.

In the construction phase, contract contingency funds provided for an additional 0.15-ft AC thickness, if needed. In remote areas where significant amounts of aggregate processing are required, separate bid items may be needed for this procedure. Pavement deflection measurements on the incremental sections would determine if any additional AC thickness is required to satisfy the tolerable deflection requirements for the conventional 10- or 20-year design period. Figures 5 and 6 from California Test 356 provide the criteria for this procedure. Figure 5 presents the relationships between pavement thickness, loading, and deflection level at or below which premature fatigue cracking will be precluded. Figure 6 is utilized to determine the amount of additional material (AC) needed to reduce the measured deflection to a tolerable level.

Sampling and Testing

The initial field study was conducted by the Transportation Laboratory during construction in October 1986. The purpose of the field study was to evaluate the incremental procedure by comparing deflection measurements on all structural layers at several test sites. The field study consisted of (1) measuring surface deflections on the asphalt concrete pavement; (2) determining in-situ density and moisture content of the base, subbase, and basement soil; and (3) collecting samples of these materials for laboratory testing. Surface deflections were measured using the Dynaflect at 0.01-mile intervals on each layer of the structural section except the ATPB. Deflection measurements were obtained over all test sections shown in Figure 4.

After AC surface deflections were measured, one site was chosen in each test and control section to measure in-situ conditions and to collect material samples including "undisturbed" subgrade samples. Sampling sites were judged to be representative of each control and test section based on deflection measurements. All laboratory and field tests were performed following standardized test procedures (*13, 14*).

In-situ material density and moisture content were measured at each sampling site. Density of each layer was measured using a nuclear gage. AC density was measured by backscatter whereas direct transmission was used to determine density of base, subbase, and basement soil. Moisture content was determined by oven drying. Test data for each layer are shown in Table 1. Density shown for AC is the mean of five or more measurements at and near each sampling site.

Material samples were collected at all sites for classification and evaluation of material properties. Extensive laboratory testing of samples was performed to fully characterize the behavior of materials in each test section. In this way, differences in deflection measurements from one section to another could be evaluated in terms of material characteristics as well as structural section variations. Subgrade soils were classified by sieve and hydrometer analysis as well as plasticity index (PI). Typical subgrade material is classified as a sandy silty

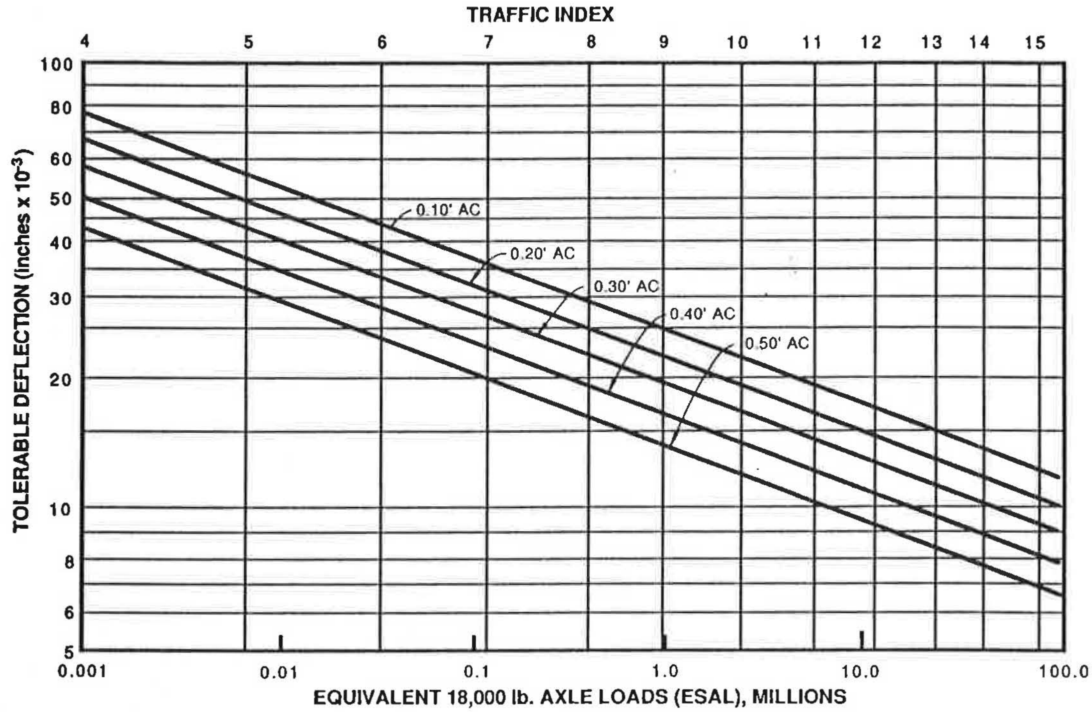

FIGURE 5 Relationships between pavement thickness, loading, and deflection level at or below which premature fatigue cracking will be precluded.

clay (CL, AASHO A-6) with low to medium plasticity, medium to high dry strength, firm and damp in place. An exception to this is the subgrade sample for Station 198, in the northern control test section. Subgrade soil at that site is a silty clay (AASHO A-7-6) characterized by higher compressibility and greater volume change than other samples. This classification is based on a PI of 23, liquid limit of 42, and more than 89 percent passing a No. 200 sieve. Higher-than-normal deflections, which were measured in the incremental and northern control sections during December 1986, are probably attributable to this more moisture-sensitive clay subgrade.

Grading analysis was performed on all base and subbase material samples. Base material samples conform to Class 2 aggregate described in Caltrans Standard Specifications (*15*). Subbase material grading met specifications for Class 2 aggregate subbase.

Basement soil properties were evaluated by triaxial compression and resilient modulus (M_r) tests. Triaxial tests were performed on at least one subgrade sample from each sampling site. Unconsolidated-undrained (UU) triaxial tests were used to determine in-situ shear strength of the subgrade soil. Subgrade cohesion was typically 14–18 psi, and the angle of internal friction varied from 16° to 24°. The strength properties of the subgrade soil at Station 198 exhibited lower strength compared to typical values at the other sampling sites. These differences correspond to the damper and more moisture-sensitive clay at the northern control section.

Resilient modulus testing was performed by the Transportation Laboratory on material from each structural layer. Subgrade cores measuring 4 in. by 8 in. were extracted from the southern control section (Station 172 + 50) and the subgrade enhancement test section (Station 177). Several base, subbase, and subgrade samples were also tested at the University of California, Berkeley (UCB), for interlaboratory comparison of test procedures, equipment, and results. M_r was determined according to AASHTO T274-82 (*14*). This test is usually performed on granular and cohesive soils. ATPB samples were also tested using the AASHTO procedure to determine M_r. Modulus of the AC was determined using the diametral test (*14*).

Results for base and subbase materials show moduli that are dependent on bulk stress (sum of principal stresses, θ) similar to research reported by Hicks (*16*). This dependence is represented by a log-linear relationship shown as $M_r = 6{,}000\ \theta^{0.5}$. Test results from UCB compared favorably with those determined at the Transportation Laboratory. Subgrade soil from the sampling sites did not exhibit typical deviator stress sensitivity as documented in the literature (*17–19*). On the basis of test results, a fixed resilient modulus of 20,000 psi was determined to be reasonable.

The subgrade M_r is much higher than the modulus used for design. The higher measured values probably reflect the lower subgrade moisture content during October, when samples were collected. Test results for the ATPB showed erratic M_r/bulk stress relationships similar to behavior described by Monismith and McLean (*20*). The variability of the ATPB moduli can be attributed to temperature sensitivity of the material. A fixed modulus of 120,000 was determined to be representative for the ATPB. On the basis of diametral modulus tests of pavement cores, a dynamic modulus of 250,000 is reasonable for the AC. The relatively low M_r for the AC is most likely attributable to the short interval between placement and compaction of the AC course and subsequent coring.

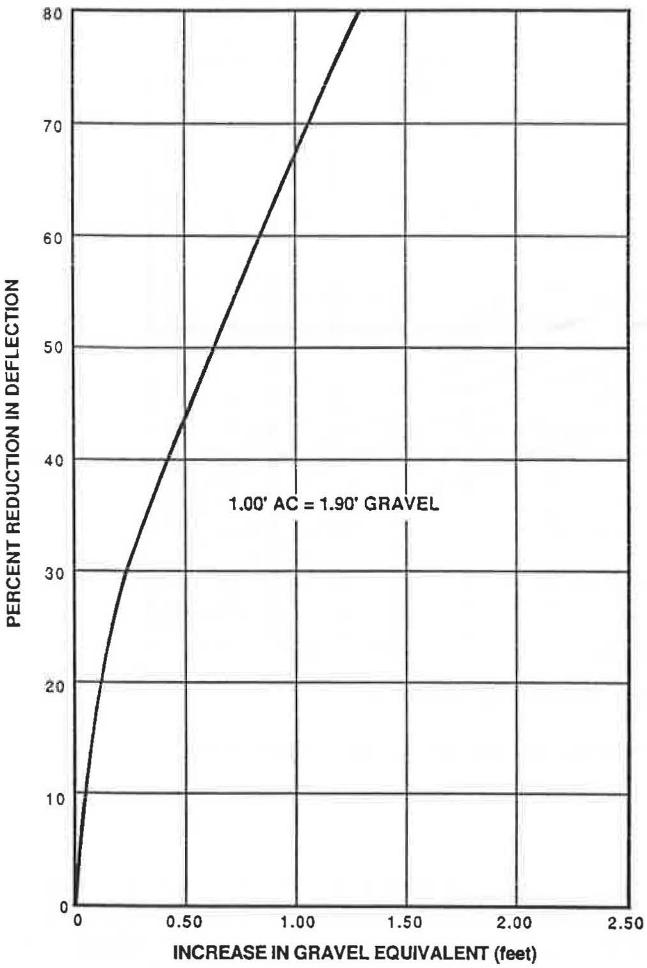

FIGURE 6 Reduction in deflection resulting from pavement overlays.

AASHTO Procedure

Structural section layer thicknesses for the test project were determined using an AASHTOWARE Software Program DNP s86/PC, v2.0 (© AASHTO 1986). The AASHTO design assumptions are shown in Table 2 (*21*). Results are shown for comparison in Table 3. An effective subgrade modulus of 6,000 psi was assumed to reflect seasonal variations in subgrade moisture at the site.

The resulting structural section thicknesses for the 10- and 20-year AASHTO design levels are considerably less than those required by the California *R*-value procedure and slightly thicker than the incremental design (see Figure 4). This is undoubtedly the result of the drainage coefficients recently adopted by AASHTO, which permit significant reduction in structural section thickness for positively drained pavement structural sections.

Mechanistic Procedure

Two methods were used to design structural sections for a 10-year and 20-year design life. The first is the Asphalt Institute procedure, which is a simplified user-friendly approach to design (*22*). Fatigue curves, granular material properties, and critical strains are implicit in this method (*23*). The other mechanistic procedure relies on the same fatigue curves as in the Asphalt Institute method. However, strains were predicted using a primary response model, PSAD, to determine layer thicknesses.

Both methods allow the engineer to determine layer thicknesses based on critical strains. The design thicknesses must satisfy two separate strain criteria, which are the horizontal tensile strain at the bottom of the AC pavement and the vertical compressive strain at the top of the subgrade. Both procedures require the number of equivalent 18-kip single-axle loads (ESALs) and material properties

The Asphalt Institute procedure requires three inputs: ESALs, subgrade resilient modulus, and types of base and surfacing. For the 3-Sac-99 project, structural elements of the design section are AC over untreated aggregate base (including subbase) supported by a weak subgrade. Design M_r for the subgrade was 4,000 psi, which is based on the relationship between M_r and *R*-value described in the Asphalt Institute Design Manual (*22*). This subgrade modulus is lower than the

TABLE 1 MATERIAL DENSITY AND MOISTURE CONTENT AT SAMPLING SITES

Structural Layer	Control Section Sta. 172+50	Fabric Section Sta. 177	Incremental & Fabric Section Sta. 182+50	Incremental Design Section Sta. 188	Control Section Sta. 198
Asphalt Concrete	2.19 gm/cc	2.26 gm/cc	2.16 gm/cc	2.25 gm/cc	2.29 gm/cc
Aggregate Base	2.21 gm/cc	2.21 gm/cc	2.19 gm/cc	2.21 gm/cc	2.27 gm/cc
	2.9%	3.6%	3.4%	2.9%	3.1%
Aggregate Base	2.31 gm/cc	2.23 gm/cc	2.25 gm/cc	2.22 gm/cc	2.24 gm/cc
	5.2%	4.8%	4.4%	3.5%	4.0%
Subgrade	2.05 gm/cc	1.97 gm/cc	2.05 gm/cc	1.95 gm/cc	1.97 gm/cc
	14.9%	14.4%	13.9%	14.3%	19%

TABLE 2 AASHTO DESIGN ASSUMPTIONS (*21*)

Structural Section Layer	Layer Coefficient (a_i)	Modulus (psi)	Drainage Coefficient (m_i)	Design Level/Traffic 10-Year TI = 11.5 $^*W_{18}=8\times10^6$	20-Year TI = 12.5 $W_{18}=16\times10^6$
AC	0.44	4.5×10^5	---	**SN=1.83	SN=2.06
ATPB	0.14	1.2×10^5	1.4	SN=0.59	SN=0.59
AB	0.13	---	1.1	SN=1.46	SN=1.46
AS	0.09	---	1.1	SN=1.67	SN=1.99
Basement Soil***	---	6×10^3	---	---	---
			Total (SN) =	5.55	6.10

Other Assumptions

Design Reliability = 95%

Overall Standard Deviation = 0.35

Terminal Serviceability Level = 2.5

*W_{18} = ESAL's

** SN = Structural Number

*** M_{re} = Effective Modulus

value used in the AASHTO design because converting *R*-value to M_r in the Asphalt Institute method does not consider drainage and seasonal adjustments, which are included in the AASHTO procedure (*21*). Table 3 shows designs using the Asphalt Institute method. Sections for both design periods require relatively thick AC layers over a relatively thin base, which is characteristic of the Asphalt Institute procedure (*23*).

The designs from the Asphalt Institute method cannot be compared directly to the sections designed using the other procedures since this method does not consider an ATPB layer. To compensate for this, a primary response model was used that included a structural element of ATPB. Use of a primary response model also allowed direct computation of strains based on M_r values from laboratory materials tests. Primary response was determined using the PSAD model developed at the UCB. The model computes strains by iteration solving M_r/stress level using tested materials properties and the CHEV5L elastic-layer algorithms (*24*). Critical strains were determined from the fatigue curves that are used implicitly in the Asphalt Institute method. A balanced design for each design life was determined by maximizing both strain criteria without exceeding the limits. Table 3 presents designs using this approach. The AC layers are noticeably thinner and the base layers are thicker than those resulting from the Asphalt Institute method. It is reasonable to conclude that an acceptable mechanistic design lies between the values shown in Table 3.

FIELD PERFORMANCE

Plots showing the results of Dynaflect deflection testing are presented in Figures 7 and 8. Figure 7 illustrates maximum deflections (Sensor No. 1) obtained at 50-ft intervals in the outer wheel track of lane 1 over individual layers of the structural section during construction. No deflection measurements were obtained directly on the ATPB. Figure 8 illustrates three sets of measurements obtained on the AC surfacing in the outer wheel track of the southbound number 2 lane (outer lane). All deflection measurements on the AC surfacing in Figures 7 and 8 have been adjusted for temperature by normalizing data to 70°F based on the Asphalt Institute procedure (*25*).

The October 1986 deflections on the 0.35-ft AC surfacing were utilized as the initial decision tool to evaluate the incremental procedure during construction, i.e., to determine if the deflection levels were at or below the tolerable level. In order to determine tolerable deflections it was first necessary to convert Dynaflect deflections to equivalent Deflectometer deflections using Figure 9. The resulting adjusted mean ($\bar{x}$) and evaluated (80th percentile) deflections along with tolerable deflection levels are presented for each test section in Table 4. These data revealed deflection levels somewhat higher than the tolerable level, suggesting the need for an additional increment of AC to satisfy the 10-year design criteria. However, a decision was made to accept the design as constructed, without any additional AC placement, anticipating a further

TABLE 3 STRUCTURAL SECTION DESIGNS

10-Year Design Layer Thickness (feet)

		Empirical		Mechanistic	
	Incremental	R-value	AASHTO	A.I.	PSAD
Asphalt Concrete	0.35	0.60	0.35	1.15	0.58
Asphalt-Treated Permeable Base	0.25	0.25	0.25	---	0.25
Aggregate Base	0.85	0.85	0.85	1.5	2.08
Aggregate Subbase	1.15	1.25	1.41		
Total SN	5.27	6.71	5.55	6.80	6.67

20-Year Design Layer Thickness (feet)

	Empirical		Mechanistic	
	R-value	AASHTO	A.I.	PSAD
Asphalt Concrete	0.65	0.39	1.29	0.83
Asphalt-Treated Permeable Base	0.25	0.25	---	0.25
Aggregate Base	0.85	0.85	1.5	1.75
Aggregate Subbase	1.50	1.67		
Total SN	7.26	6.10	7.54	7.51

SN = Structural Number

reduction in deflection level due to age stiffening of the asphalt concrete, with provision for monitoring pavement performance by subsequent deflection surveys. Also, the final surface layer of 0.06-ft open graded AC would still be placed. The control sections would also receive an additional 0.30 ft of dense graded AC, plus 0.06 ft open graded AC, as scheduled, after the December 1986 deflection survey. The deflection levels of the various sections were found to be quite uniform at the time of the October 1986 survey. Results of surveys in December 1986 and June 1987 reflect additional densification due to traffic compaction, age stiffening of the AC, changes in subgrade moisture, and deflection attenuation of the control sections due to placement of additional AC.

The temperature-adjusted June 1987 deflections are all below the tolerable level (Table 5). Note that the incremental and control sections were designed using a 5-*R*-value basement soil. In actuality, samples taken from the subgrade at two locations produced *R*-value test results of 14 at Station 186 and 40 at Station 200. This probably indicates that the imported material was carried into the experimental area and was of higher quality than 15 *R*-value. This also suggests conservatism in the original *R*-value design. Future deflection measurements will provide a better overall evaluation of seasonal variations as the different sections reach equilibrium.

DISCUSSION OF RESULTS

The June 1987 deflection measurements were surprisingly low on all test sections, considering that they were made in the late spring on a relatively soft subgrade.

The basis for this comment is an earlier research study on this portion of Route 99 which was carried out between 1959 and 1969. The goal of that study was to establish the deflection attenuation properties of asphalt concrete, cement-treated base, and aggregate base (*26*). The original structural section consisted of 0.08-ft armor coat and 0.50-ft aggregate base over 0.75-ft aggregate subbase.

Three test sections were overlain with 0.05-ft open-graded asphalt concrete, 0.25-ft asphalt concrete, 0.25-ft asphalt concrete base and 0.50-ft aggregate base. The total gravel equivalence of the original structural section plus the overlay was 2.99 ft, which is close to 3.11 ft for the incremental sections. Spring deflection measurements over the completed roadway averaged 0.029 in. in contrast to 0.014 in. for the incremental sections.

Similarly, four test sections were overlain with 1.00-ft aggregate base, 0.25-ft asphalt concrete, and 0.05-ft open-graded asphalt concrete. These, plus the original structural section, were almost identical in gravel equivalence to the incremental

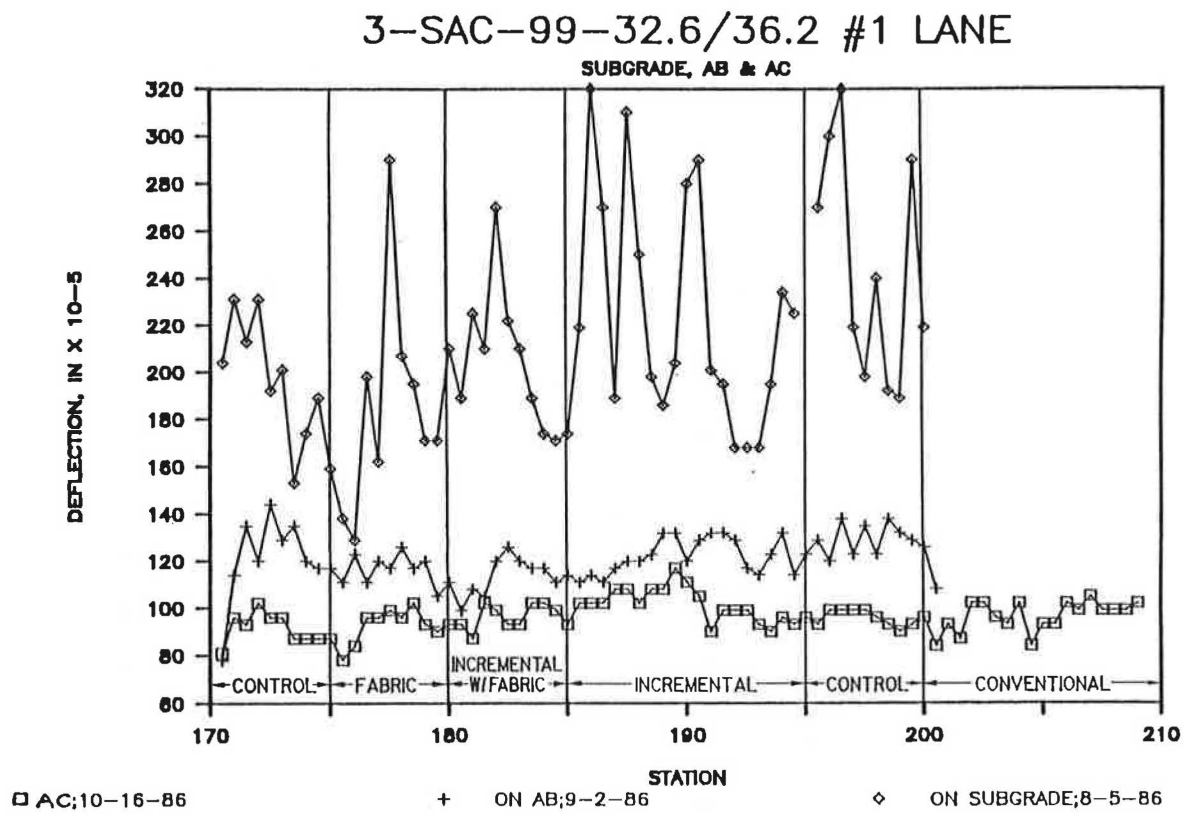

FIGURE 7 Dynaflect deflections on subgrade, base, and AC.

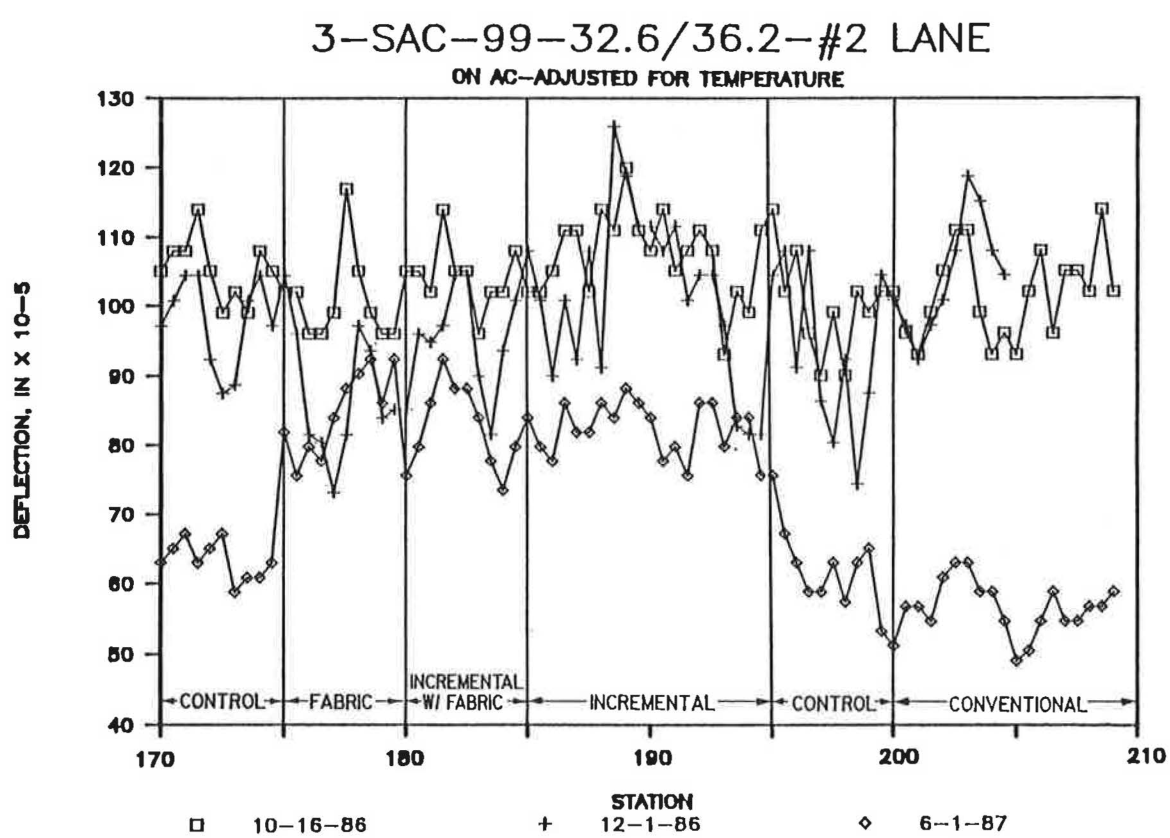

FIGURE 8 Dynaflect deflections on AC adjusted for temperature.

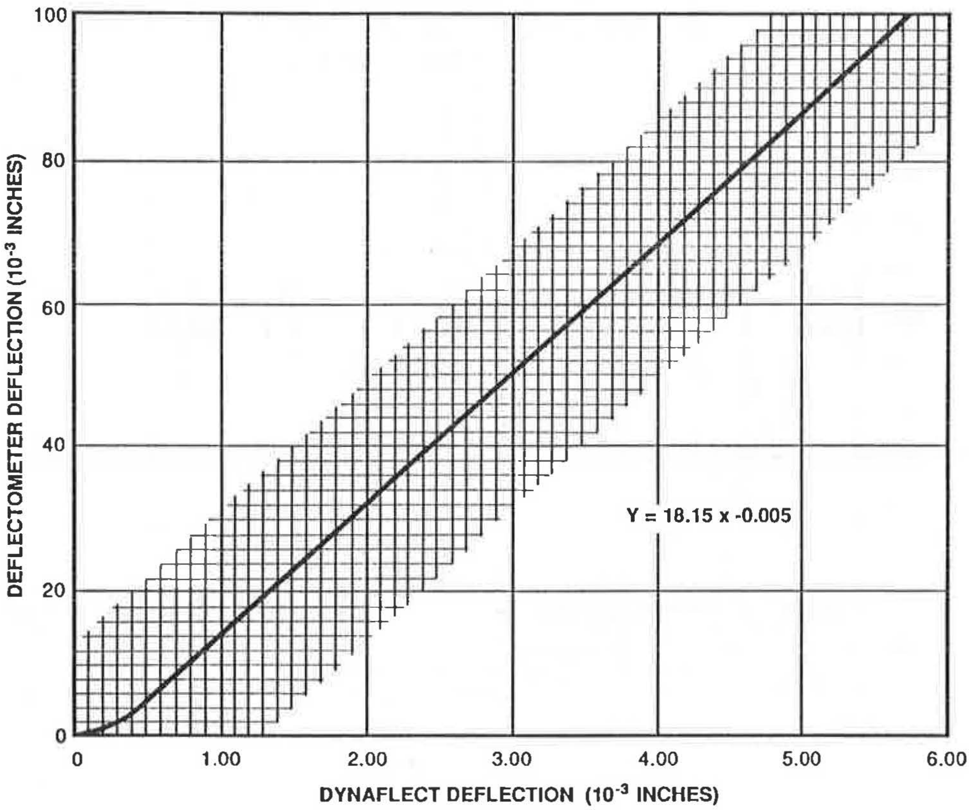

FIGURE 9 Comparison of Dynaflect and deflectometer deflections.

TABLE 4 DEFLECTION MEASUREMENTS TO EVALUATE INCREMENTAL DESIGN

	Deflection Measurements (10^{-3} Inches)				Tolerable Deflectometer** Deflection (10^{-3} Inches) Design Level/Traffic Index		
	Dynaflect		Equivalent Deflectometer				
Test Section	$\bar{x}$*	80%tile	$\bar{x}$	80%tile	5 yrs/10	10 yrs/11.5	20 yrs/12.5
Control Section	1.05	1.09	14	15	15	12	11
Subgrade Enhancement Fabric Section	1.01	1.07	13	14	"	"	"
Incremental Design with Subgrade Enhancement Fabric	1.04	1.08	14	15	"	"	"
Incremental Design	1.07	1.12	14	15	15	12	11
Control Section (Sta. 195 to 200)	1.01	1.08	13	15	"	"	"
Conventional Section with Lime-Treated Subgrade (LTS)	1.02	1.07	14	14	"	"	"

* $\bar{x}$ = mean deflection.

** Deflections tolerable for 0.35 foot of AC surfacing.

TABLE 5 MEAN AND TOLERABLE DEFLECTION MEASUREMENTS ON AC SURFACING

Test Section	Mean Deflection Level (10^{-3} Inches)*						Tolerable Deflectometer** Deflection (10^{-3} Inches)		
	Dynaflect			Equivalent Deflectometer			Design Level/Traffic Index		
	10/16/86	12/1/86	6/1/87	10/16/86	12/1/86	6/1/87	5 yrs/10	10 yrs/11.5	20 yrs/12.5
Control Section (Sta. 170 to 175 0.65 foot AC)	1.05	0.98	0.62	14	13	6	10	9	8
Subgrade Enhancement Fabric Section (0.35 foot AC)	1.01	0.88	0.84	13	11	10	15	12	11
Incremental Design with Subgrade Enhancement Fabric (0.35 foot AC)	1.04	0.95	0.81	14	12	10	15	12	11
Incremental Design (0.35 foot AC)	1.07	1.02	0.81	14	14	10	15	12	11
Control Section (Sta. 195 to 200 0.65 foot AC)	1.01	0.94	0.64	13	12	7	10	9	8
Conventional Section with Lime-Treated Subgrade (LTS) (0.65 foot AC)	1.02	1.04	0.61	14	14	6	10	9	8

* Corrected for temperature

** Deflection tolerable for AC thickness shown in parentheses for each test section.

sections (3.07 ft). Spring deflection measurements over the completed roadway averaged 0.026 in., which is almost double those on the incremental sections. Because of the uniform nature of the subgrade and the fact that the area is subject to irrigation, one would not expect to find this degree of variation in the response of nearly equivalent structural sections.

The surprisingly low level of deflection on the incremental sections is very possibly the result of the positive rapid drainage feature common to all test sections, including the controls. Although, as would be expected, control section deflection levels were substantially lower than those of the incremental sections, the relatively high strength of all test sections masked the effect of lime stabilization or subgrade enhancement with geotextiles even though an earlier limited study (*11*) on the same subgrade revealed that the use of geotextiles was equivalent to 0.58 ft of aggregate base in terms of deflection attenuation. In retrospect, construction of equivalent but undrained test sections in the experiment would have proven extremely informative.

The decrease in deflection level on all sections between October and June represents the frequently observed effect of traffic compaction and age stiffening of the asphalt concrete pavement. All sections, however, exhibited deflection levels that would preclude premature fatigue cracking for average asphalt concrete pavements in California.

CONCLUSIONS

The results of laboratory and field measurements and tests on the six test sections on Route 03-Sac-99 justify the following conclusions:

1. For the assumed traffic, subgrade conditions, and pavement section material characteristics at the test site, the mechanistic-empirical and one empirical (California) procedure indicate the need for a thicker structural section than was found to be necessary using the incremental procedure.
2. The structural section indicated by the recently modified

AASHTO (empirical) procedure for the 10-year design was relatively consistent with that resulting from the incremental procedure. This is probably due to drainage coefficients, which apparently are reasonable for this site.

3. The deflection measurements on all test sections were uniformly low although they are consistent with structural section strength and thickness characteristics. This is attributed to the positive rapid drainage features used on all test sections.

4. The late spring deflection measurements reflected the normal effects of traffic compaction and age stiffening of the asphalt concrete surfacing. For the incremental sections, they confirmed that the second increment of asphalt concrete surfacing was not required to avoid premature fatigue cracking of the surfacing.

5. The incremental procedure can reduce thickness requirements by providing a way to quantify the effects of favorable subgrade materials variations and rapid positive drainage of the pavement structural section. At this particular site, a structural section cost savings of 18 percent for a 10-year design would have been realized with comparable fatigue performance of the AC surfacing.

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REFERENCES

1. C. K. Kennedy. Analytical Flexible Pavement Design: A Critical State of the Art Review 1984. *Proc., Institution of Civil Engineers,* Vol. 78, Part 1, 1985, pp. 897–917.
2. G. B. Sherman. *Recent Changes in the California Design Method for Structural Sections of Flexible Pavement.* California Division of Highways, Materials and Research Department, Sacramento, 1958.
3. E. Zube, C. G. Gates, and D. R. Howe. *A Report on an Investigation Concerning the Effect of Lowering the Exudation Pressure Interpolation Point in the R-Value Test.* California Division of Highways, Materials and Research Department, Sacramento, 1959.
4. F. N. Hveem. *Thickness of Flexible Pavements by the California Formula Compared to AASHO Road Test Data.* California Division of Highways, Materials and Research Department, Sacramento, 1963.
5. Drainage of the Structural Section. *Highway Design Manual,* Topic 606. California Division of Transportation, 1987.
6. R. L. Moore. *Comparison of AC Pavement Structural Deficiencies Calculated Using Deflection Measurements and the R-Value Procedure.* Transportation Laboratory, California Department of Transportation, Sacramento, 1981.
7. F. N. Hveem. Pavement Deflections and Fatigue Failures. *Bulletin 114,* HRB, National Research Council, Washington, D.C., 1955.
8. D. V. Roberts, J. B. Skog, J. A. Matthews, W. Mann. *Statewide Flexible Pavement Performance and Deflection Study.* Report No. FHWA-CA-TL-78-28. Transportation Laboratory, California Department of Transportation, Sacramento, 1978.
9. W. Mann, J. A. Matthews, and J. T. Webster. *An Evaluation of the Current California Method to Determine AC Overlay Thickness.* Report No. FHWA-CA-TL-80-28. Transportation Laboratory, California Department of Transportation, Sacramento, 1980.
10. C. R. Foster. *Proc., 2nd International Conference on the Structural Design of Asphalt Pavements.* University of Michigan, Ann Arbor, 1962, p. 13.
11. R. L. Moore. *Evaluating the Use of Reinforcing Fabric to Support the Pavement Structural Section.* Memo Report 641139. Transportation Laboratory, California Department of Transportation, Sacramento, 1983.
12. R. L. Moore. *Geotextiles as Pavement Structural Section Elements.* Memo Report 643380. Transportation Laboratory, California Department of Transportation, Sacramento, 1985.
13. *Manual of Test,* 3rd Ed., Vols. I, II, and III. Transportation Laboratory, California Department of Transportation, Sacramento, 1978.
14. *Standard Specifications for Transportation Materials and Methods of Sampling and Testing,* Part II, 14th Ed. AASHTO, 1986.
15. *Standard Specifications.* California Department of Transportation, Sacramento, 1984.
16. R. G. Hicks. *Factors Influencing the Resilient Properties of Granular Materials.* Ph.D. dissertation. University of California, Berkeley, 1970.
17. R. D. Barksdale and R. G. Hicks. Material Characterization and Layered Theory for Use in Fatigue Analyses. In *Special Report 140.* HRB, National Research Council, Washington, D.C., 1973.
18. D. E. Newcomb. *A Review of Pavement Analysis Computer Programs—Interim Report No. 1.* University of Washington, Seattle, 1984.
19. R. G. Hicks. *Use of Layered Theory in the Design and Evaluation of Pavement Systems.* Alaska Department of Transportation and Public Facilities, Fairbanks, 1982.
20. C. L. Monismith and D. B. McLean. *Design Considerations for Asphalt Pavements.* Report No. TE 71-8. Institute of Transportation and Traffic Engineering, University of California, Berkeley, 1971.
21. *AASHTO Guide for Design of Pavement Structures.* American Association of State Highway and Transportation Officials, Washington, D.C., Chap. 2, 1986.
22. *Thickness Design—Asphalt Pavements for Highways and Streets.* Manual Series No. 1 (MS-1). The Asphalt Institute, College Park, Md., 1981.
23. *Research and Development of the Asphalt Institute's Thickness Design Manual (MS-1),* 9th Ed. Research Report No. 82-2 (RR-82-2). The Asphalt Institute, College Park, Md., 1982.
24. H. Warren and W. L. Dieckmann. *Numerical Computation of Stresses and Strains in a Multiple-Layer Asphalt Pavement System.* Chevron Research Corp., 1963.
25. *Asphalt Overlays for Highway and Street Rehabilitation.* Manual Series No. 17. The Asphalt Institute, 1983, p. 39, Fig. IV-2.
26. G. G. Sherman, C. G. Gates, M. M. Hatano, and D. L. Durr. *Deflection Studies Performed on El Centro Road (03-Sac-99) Ten-Year Report.* California Division of Highways, Materials and Research Department, 1970.

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