CALTRANS ACCELERATED PAVEMENT TEST (CAL/APT) PROGRAM—TEST RESULTS: 1994–1997

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Introduction

The Caltrans Accelerated Pavement Testing (CAL/APT) Program, a research and development activity, is a joint effort between Caltrans (the California Department of Transportation), the University of California at Berkeley (UCB), the Division of Roads and Transport Technology of the Council of Scientific and Industrial Research (CSIR) of South Africa, and Dyntatest Consulting Inc. of Ojai, California.

The program utilizes two Heavy Vehicle Simulators (HVS) developed in South Africa. One of the HVS units is used to test full-scale pavements in a controlled environment at UCB's Richmond Field Station (RFS) while the other is utilized for testing in-service pavements. An extensive laboratory testing program involving the laboratories of both UCB and Caltrans complements the full-scale accelerated loading testing. Figure 1 illustrates a simplified framework within which the program operates.

This paper discusses results obtained to date from the Phase II portion of the program which was initiated July 1, 1994.[1] It includes an evaluation of results of tests on pavement sections constructed at the RFS according to Caltrans specifications coupled with associated laboratory tests and analyses. Table 1 provides a summary of the work completed to date together with references for associated publications with each of the studies listed.

Heavy Vehicle Simulator

Figure 2 provides a schematic representation of the HVS developed by the CSIR and used by Caltrans (two units purchased from the CSIR).

Wheel loads of up to 200 kN (45,000 lb) are applied on a half axle using dual, standard-size truck tires or a single aircraft tire moving in either a uni- or bi-directional mode at speeds up to 8 km/hr (5 mph). At this rate, approximately 18,000-load repetitions are applied per day in the bi-directional mode. Longitudinal wheel travel is 8.0 m (31.5 ft) and lateral travel is programmable over 1.5 m (4.9 ft).

Once an HVS is at a test location, its full mobility enables it to move under its own power in maneuvering to nearby test sections. Pavement instrumentation used routinely includes multi-depth deflectometers, surface profilometer, road surface deflectometer, crack activity meter, thermocouples, photographic surface crack detector equipment, and a nuclear density gauge.

Pavement Sections

The pavement structures tested thus far at the RFS are shown in Figure 3. They consist of a clay subgrade (AASHTO A-7-6), aggregate subbase (ASB), aggregate base (AB), asphalt treated permeable base (ATPB), and asphalt concrete (AC). Thicknesses for the pavement sections were selected according to the Caltrans design procedure for a Traffic Index of 9

[1] Phase 1 was accomplished in the period of April 1993–January 1994

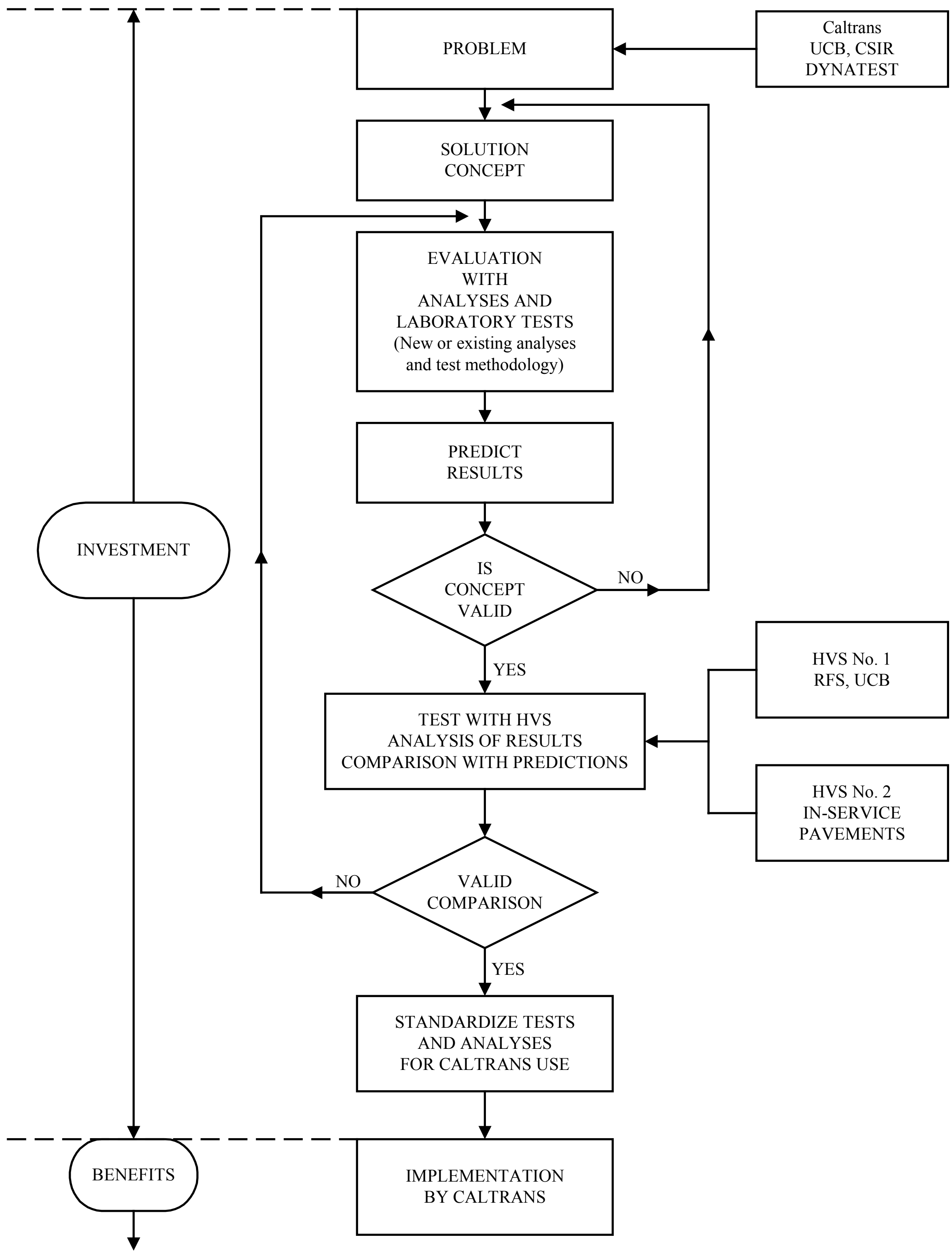


Figure 1 CAL/APT Framework

Table 1

Study	Type	Objectives(s)	Results	References
(1) Fatigue performance of asphalt concrete mixesCrelationship to pavement performance in California.	Laboratory study, analyses	• Evaluate effects of asphalt content and air-voids content on fatigue response of a typical California asphalt concrete mix. • Demonstrate usefulness of SHRP-developed procedure for mix and pavement analysis and design to achieve improved fatigue performance.	• Used in interpretation of data obtained in Study (2). • Used in comparative analysis in Study (6). • Used in pay-factor Study (7). • Recommendation for use of "rich-bottom" design.	*(1), (2)*
(2) Accelerated loading on four full-scale pavements with untreated aggregate and asphalt-treated permeable bases.	Accelerated pavement tests with HVS, laboratory tests, analyses	• Primary objective—develop data to quantitatively verify existing Caltrans design methodologies for asphalt treated permeable base (ATPB) pavements and conventional aggregate base pavements with regard to failure under traffic at moderate temperatures. • Other objectives— · quantify effective elastic moduli of various pavement layers · quantify stress dependence of materials in pavement layers · determine failure mechanisms in various layers · determine and compare fatigue lives of the two types of pavement structures	• Importance of mix compaction conclusively demonstrated. Recommendation for "tightening" Caltrans compaction requirements. • Comparison of measured and predicted results demonstrate the validity of the fatigue analysis and design system developed during the SHRP program and refined within the CAL/APT program. • The lack of bond between compacted lifts of asphalt concrete observed in the HVS tests suggests re-examination by Caltrans of the use of a tack coat between lifts to improve the bond. • The subgrade strain criteria used by the Asphalt Institute can be used by Caltrans as a part of a mechanistic-empirical design procedure.	*(3), (4), (5), (6), (7), (8)*
(3) Asphalt treated permeable base study	Laboratory study, analyses	• Measure effects of water on ATPB stiffness through laboratory testing. • Relate soaking performed in laboratory and its effects on ATPB stiffness to field conditions. • Provide "bridge" between HVS tests conducted with ATPB in dry state to in-situ performance with some likelihood of ATPB being saturated. • Evaluate design philosophy behind use of ATPB and its implementation to date. • From overall evaluation, provide recommendations pertaining to use of ATPB in California.	• Improved compaction of the asphalt concrete layer and proper structural design based on results of studies (1) and (2) as well as reduced permeability of asphalt concrete resulting from improved compaction would eliminate the need for the ATPB directly beneath the asphalt concrete layer. • Because of the susceptibility of ATPB to the action of water as currently specified, an improved design is recommended using more asphalt and/or modified binders such as asphalt rubber. • To prevent clogging of the ATPB, if used, suitable filters should be incorporated in the structural section.	*(9)*

Table 1 (cont.)

(4) Mix rutting using accelerated loading at elevated pavement temperature(s)	Accelerated pavement tests with HVS, laboratory tests, analyses	• Study mix rutting under radial, bias-ply, and wide-base tires at elevated temperatures.	• Test series on ten sections demonstrated the rapidity with which the influence of different tire types on asphalt concrete rutting can be evaluated. • Results demonstrate the increased rutting which can result with wide base single tires as compared to dual tires for same total load and tire pressure under channelized traffic conditions.	Report in preparation
(5) Tire pressure study using 3D stress sensor [Vehicle-Road Surface Pressure Transducer Array (VRPSTA)]	HVS loading with different tire types, pressures, single and dual tire configurations	• Define stress distributions, both vertical and horizontal, exerted by a range in tire types, pressures, and configurations including bias-ply, radial, wide-base radial, used aircraft, radial (new and used) wide-base (off-road) radial.	• Tire pressure analysis used in finite element analysis to: · evaluate crack patterns observed in HVS tests. · provide confirmation of the results of layered elastic analysis of HVS tests. · provide a basis for the use of the simple shear test for permanent deformation evaluation of mixes.	*(10)*
(6) Comparison of AASHTO and Caltrans pavement design methods	Analyses	• Quantify differences in pavement thicknesses by two methods. • Compare predicted performance for pavement designs considered equal within Caltrans procedure. • Evaluate effect of drainage conditions on pavement structures designed by AASHTO procedure. • Demonstrate the usefulness of mechanistic-empirical design procedure.	• Provides additional evidence to Caltrans to make the transition from their current design procedure to a mechanistic-empirical procedure.	*(11)*
(7) Pay-factor study	Analyses	• Use fatigue analysis/design system (calibrated with HVS tests) to develop pay-factors for compaction control (relative density), asphalt content, and asphalt concrete thickness.	• Recommended that Caltrans uses factors on a trial basis for selected QC/QA projects currently underway (shadow projects).	*(12)*
(8) Effects of binder loss stiffness (SHRP PG binder specification) on fatigue performance of pavements	Analyses	• Use fatigue analysis/design system (calibrated with HVS tests) to evaluate effects of binder loss modulus, $G^*\sin\delta$, on pavement performance in fatigue.	• Recommendation that $G^*\sin\delta$ be eliminated from PG-specification for binders.	*(13)*

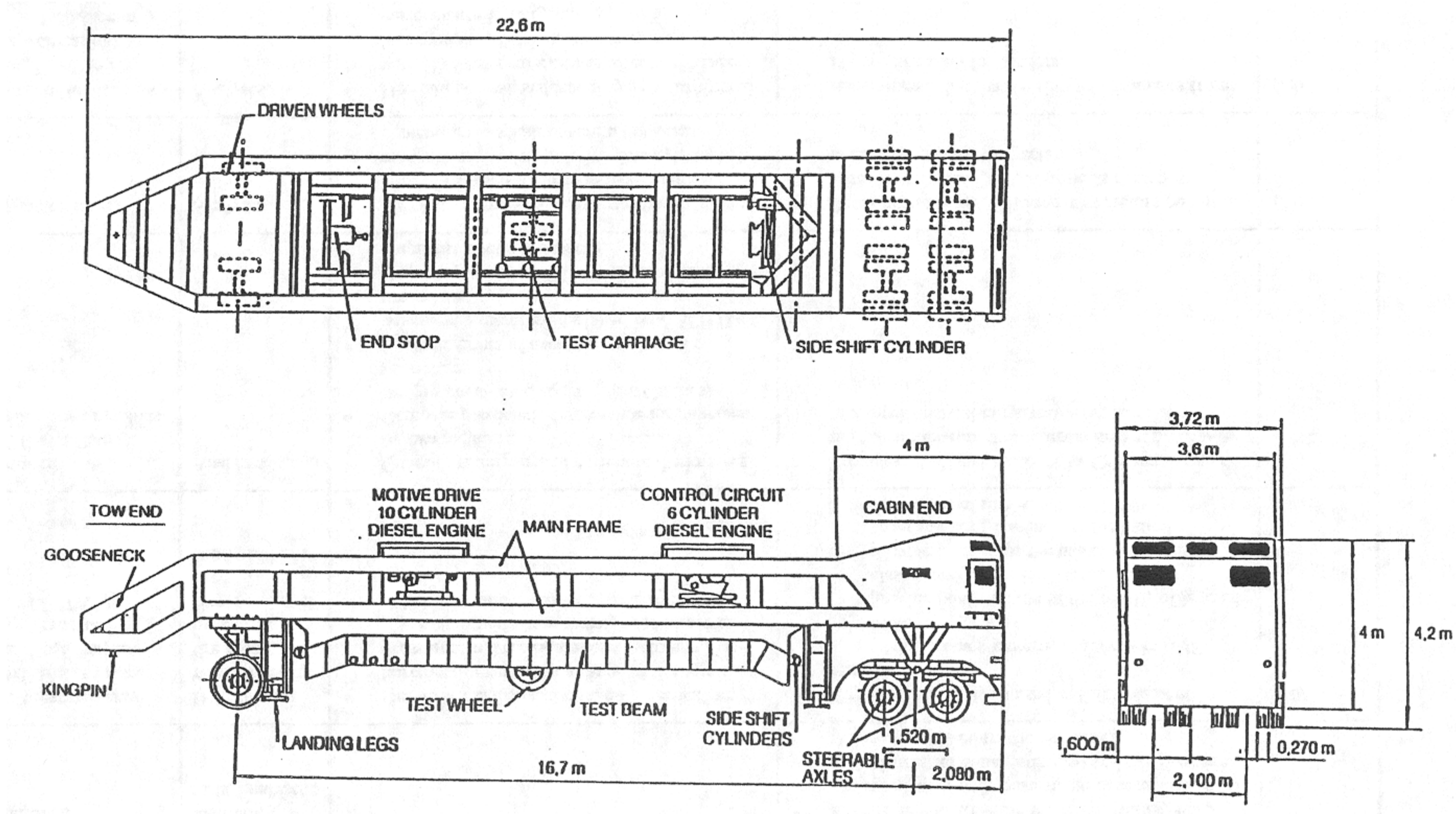


Figure 2. Line Diagram of the Heavy Vehicle Simulator (HVS)

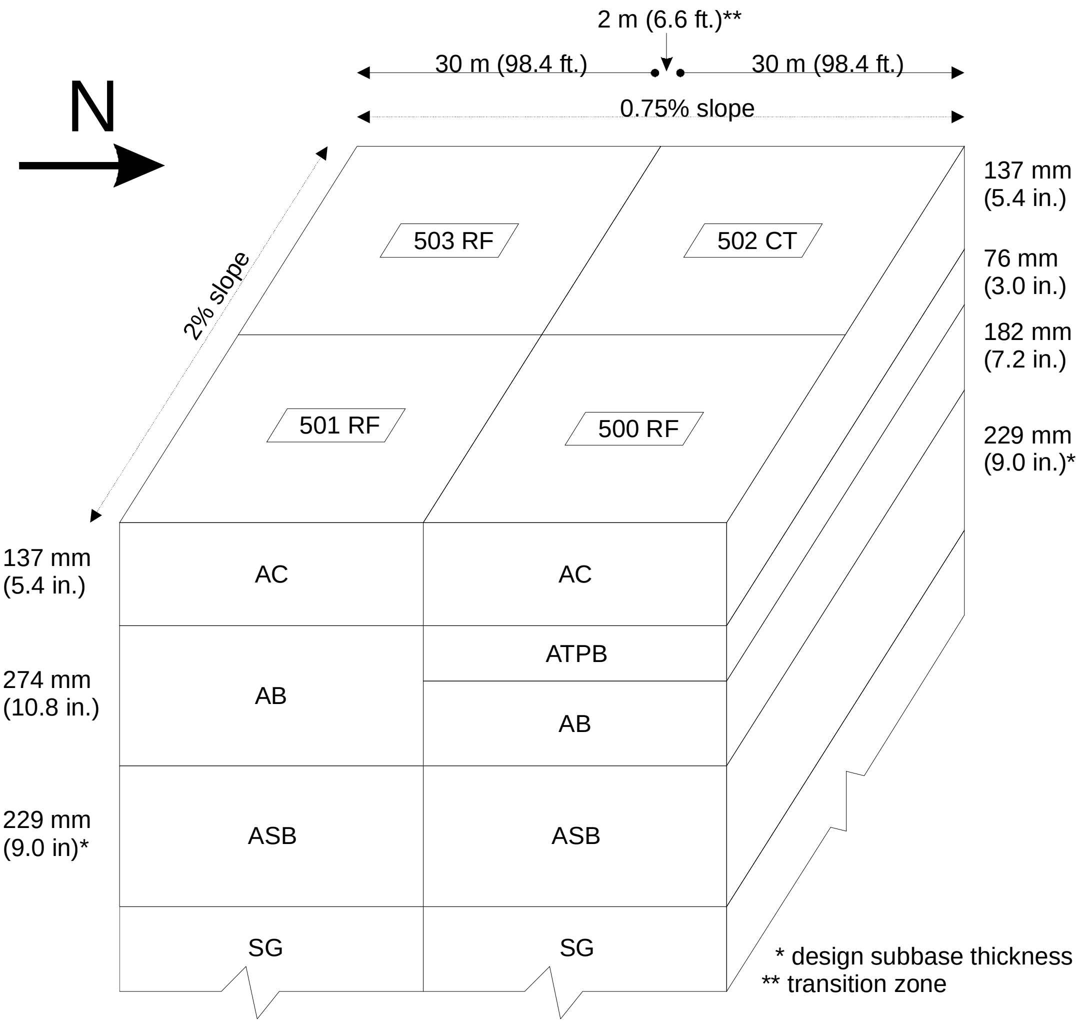


Figure 3. Test Pavements

(800,000 to 1,200,000 ESALs) and a design stabilometer "R" value for the subgrade of 10 (measured range of 4 to 30).

The test pavements were constructed by a commercial contractor according to Caltrans specifications. When the construction of the test pavements was completed (1995), cores and slabs of the asphalt concrete were taken from the pavement for testing. Due to the total required thickness of 137 mm (5.4 in), the AC was placed and compacted in two 68.5-mm (2.7-in) lifts. Table 2 lists the actual thicknesses of each of the layers in the four test sections. When the cores were obtained, a weakness in (and even a lack of) the bond between the two asphalt-concrete lifts was observed.

Following completion of the HVS tests on the four sections, overlays were placed. One overlay was a conventional dense graded asphalt concrete (DGAC) with a compacted thickness of 60-70 mm (0.20-0.25 ft) on Sections 500 and 501. The other was a gap-graded asphalt rubber hot mix (ARHM-GG) 37 mm (0.12 ft) in thickness on Sections 502 and 503. Construction of the overlays was accomplished in March, 1997. Unlike initial construction, a tack coat was applied before the overlay construction. Cores taken after this construction consistently showed good bonding between the overlay and upper lift of the original asphalt concrete.

Table 2. Design and the actual layer thicknesses (mm) of test sections

	Section 500RF		Section 501RF		Section 502CT		Section 503RF	
Layer	design	actual	design	actual	design	actual	design	actual
Upper AC lift	61	74	61	63	61	68	61	74
Lower AC lift	76	76	76	84	76	80	76	88
ATPB	76	76	N/A	N/A	76	76	N/A	N/A
AB	182	182	274	274	182	182	274	274
ASB[a]	229	137	229	215	229	215	229	305

[a] Because of the 2 percent cross slope, and to maintain the thickness of the other pavement components, the thickness of the subbase varied as shown herein.

Mix Design and Analysis System for Fatigue

The design and analyses system for fatigue used to interpret the results presented herein was originally developed as a part of the strategic Highway Research Program (SHRP) *(15)*, extended subsequently to efficiently treat in-situ temperatures *(18)*, calibrated to the Caltrans flexible-pavement design system *(1)*, extended to include construction variability *(12)*, and used to interpret the results of the first series of HVS tests at the RFS and described subsequently. Since it serves as a basis for a number of the studies reported herein, a brief description of the system is included in this section.

The system requires the fatigue response characteristics of the mix to be used in the analyses; it can be applied in a number of different ways, two of which are:

1. evaluation of the likelihood that the mix will satisfactorily resist fatigue cracking in the design pavement under anticipated in-situ conditions.
2. estimation of the number of ESALs that can be sustained in a design setting or in the HVS test program.

Mix response fatigue characteristics are expressed by an equation of the following form:

$$N = a\left(\frac{1}{\varepsilon_t}\right)^b$$

where N = number of load repetitions to failure under the tensile strain ε_t, and a, b = mix specific coefficients determined by test. The tensile strain at which the repetitions are estimated is determined for the representative load by multi-layer elastic analysis.

For the studies reported herein, i.e. estimation of load repetitions (expressed in terms of ESALs), the second application of the system noted above was used according to the following expression:

$$ESALs = \frac{N \bullet SF}{TCF \bullet M}$$

in which *ESALs* = the number of equivalent, 80 kN single axle loads that can be sustained in situ before failure, *N* = the number of laboratory load repetitions to failure under the anticipated in situ strain level, *SF* = a shift factor necessary to reconcile the difference between fatigue in situ and that in the laboratory, *TCF* = a temperature conversion factor which converts loading effects under the expected range of temperatures in the in situ temperature environment to those under the single temperature typically used in laboratory testing for conventional asphalts (20 C), and *M* = a reliability multiplier based on the level of acceptable risk and variabilities associated both with computing *N* and with estimating actual traffic loading.

The estimate of *N* is based on laboratory testing which measures the stiffness and fatigue life of the asphalt mixture and on elastic multilayer analysis which determines the critical strain (ε) expected at the underside of the asphalt layer in situ under the standard 80 kN axle load. *N* is simply the laboratory fatigue life that would result from the repetitive application of the strain expected in situ.

The critical strain is also used in determining the shift factor, *SF*, as follows:

$$SF = 3.1833\ C\ 10^{-5}\varepsilon^{-1.3759} \qquad \textit{for} \qquad \varepsilon \geq 0.000040$$

The shift factor accounts for differences between laboratory and field conditions, including, but not limited to traffic wander, crack propagation, and rest periods. The shift factor relation is based on calibration of laboratory results against the Caltrans pavement thickness design procedure reported in the next section.

The temperature conversion factor, *TCF*, has been determined for three representative California environments including those in coastal, desert, and mountain regions. A general expression relating *TCF* to each environment has the form:

$$TCF = a \ln(t) - b$$

in which t = asphalt concrete thickness and a and b are coefficients associated with each environment.

The reliability multiplier, M, is calculated as follows:

$$M = e^{z\sqrt{var(\ln N) + var(\ln ESALs)}}$$

in which e = the base of natural or Naperian logarithms, Z = a factor depending solely on the design reliability, var(lnN) = the variance of the logarithm of the laboratory fatigue life estimated at the in situ strain level, and var(ln ESALs) = the variance of the estimate of the logarithm of the design ESALs (i.e., the variance associated with uncertainty in the traffic estimate).

The variance in the logarithm of the laboratory fatigue life, var(lnN), is currently calculated using a Monte Carlo simulation procedure. This procedure accounts for the inherent variability in the fatigue measurements; the nature of the laboratory testing program (principally the number of test specimens and the strain levels); the extent of extrapolation necessary for estimating fatigue life at the design, in situ strain level; mix variability due to construction (namely asphalt and air-void contents); and structural variability due to construction (namely thickness of asphalt concrete surface and the equivalent stiffness of modulus of supporting layers). The calculations of design ESALs reported herein used construction variances thought to be typical of prevailing California construction practice. A var(lnESALs) of 0.3 has also been used in these calculations.

Summary and FindingsCInvestigations Completed Through June 1997

This section provides a brief summary of each of the studies undertaken together with specific conclusions and recommendations obtained from them. The concluding section that follows will synthesize these results into a coherent set of recommendations which, if implemented, would result in improved pavement performance and, potentially, substantial cost savings.

1. Fatigue Performance of Asphalt Concrete Mixes (1,2).

This study, using multilayer elastic analysis and laboratory fatigue testing, examined the influence of mix proportions, specifically asphalt and air-void contents, on fatigue behavior both in the laboratory and in situ. It refined and recalibrated a mix design and analysis system developed as a part of the SHRP asphalt research endeavor *(14)*. This system is capable of quickly and easily determining the likely fatigue endurance of design mixes in specific pavement structures, at specific locations, and under anticipated traffic loading. Of particular significance is the integration of mix and structural components and the explicit treatment of both testing and construction variablities which, in turn, permits selections of an acceptable level of risk for the pavement section. Specific results include the following:

- Construction control is an important consideration in mix design. With respect to fatigue performance, accurate control of air-voids content is much more important than accurate control of asphalt content. Complicating this matter is the likelihood that smaller-than-specified asphalt contents will result in increased air-voids contents unless compactive effect is increased to compensate.
- In-situ fatigue performance can be quite sensitive to construction variability and, by inference, the caliber of the quality assurance program. To restrict the risk of premature failure to tolerable levels, mix design and/or structural design must recognize and, if possible, compensate for expected construction practice.
- Monte Carlo simulation is an effective technique for simulating fatigue-life distributions resulting from testing, extrapolation, and/or construction variabilities. The ability to quantify fatigue-life distributions has significant potential for a) establishing rational performance requirements, b) taking actions to reduce the risk of failure to meet performance requirements, and c) establishing performance-based contractor pay schedules. Figure 4 illustrates the results of such a simulation. In this figure, the pavement section listed as 11ab20 [Traffic Index of 11 (ESAL range 4.5×10^6 – 6.0×10^6), untreated aggregate base, 20 subgrade R-value], has a fatigue life at the targeted asphalt and air-void content of approximately 4.4×10^6 ESALs; this corresponds to the median value shown in the figure. The mean fatigue life and its standard deviation are 5.4×10^6 and 3.9×10^6 ESALs, respectively.

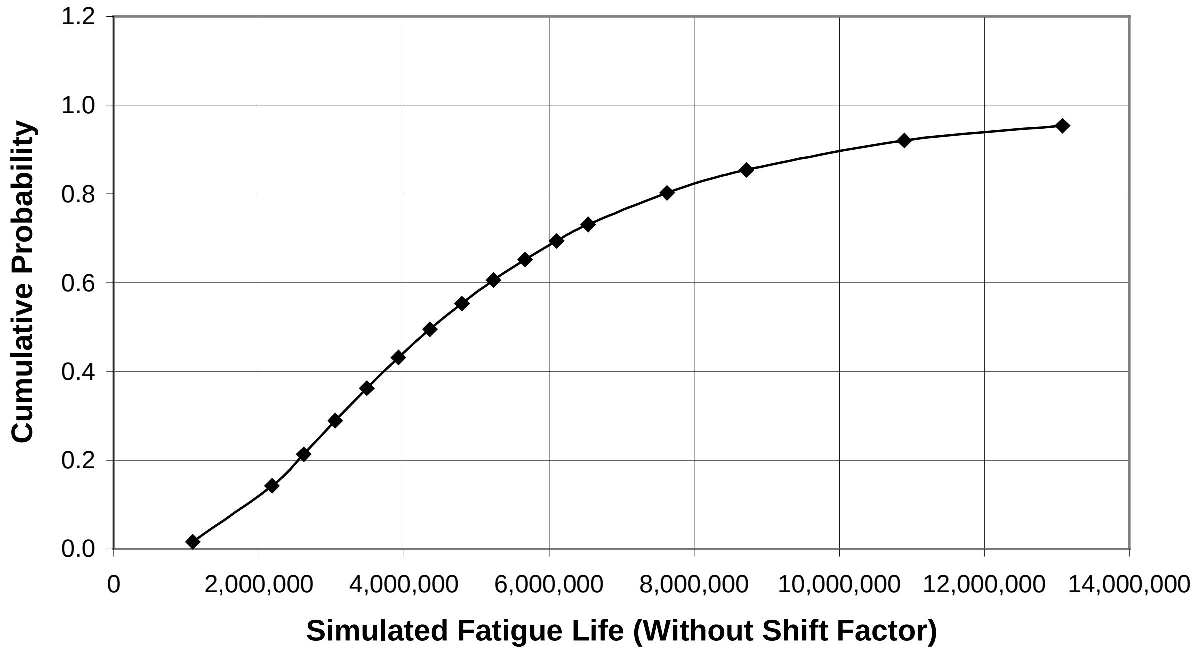
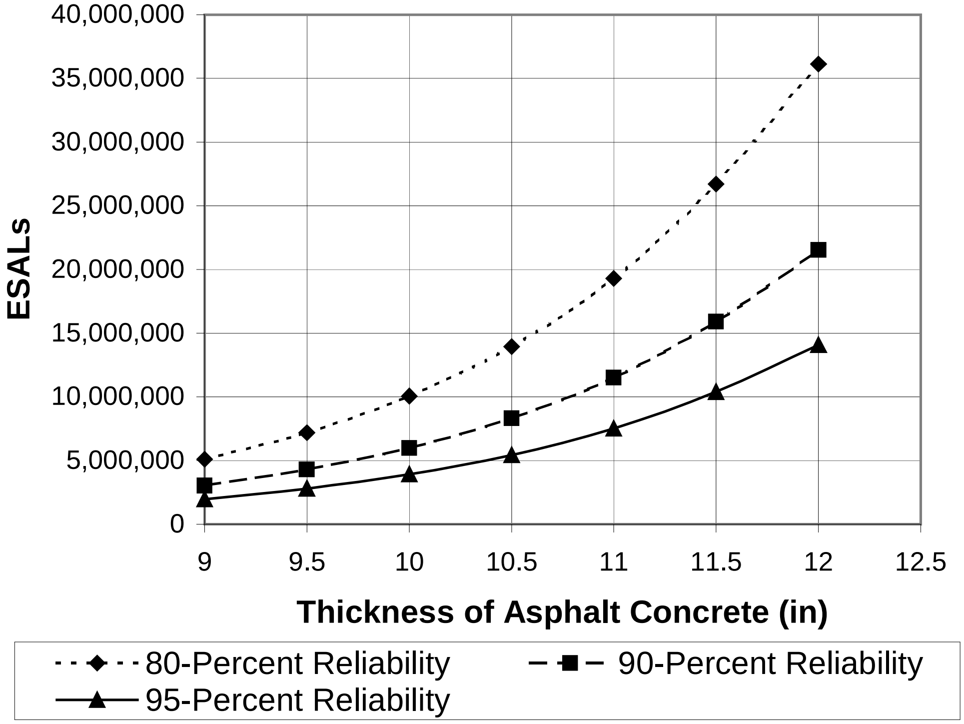


Figure 4. Dispersion of fatigue life (measured in ESALs) resulting from construction variability (11ab20)

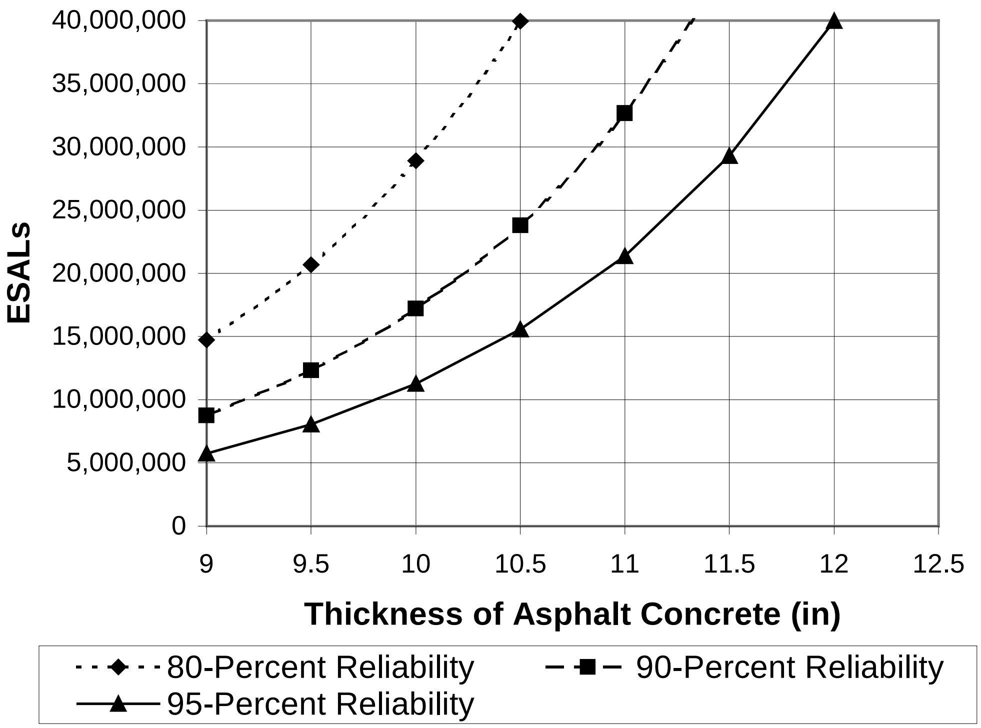
- The mix design and analysis system can be effectively used to determine the consistency of structural design procedures with respect to the control of fatigue distress. For example, conditions where California practice is most likely to yield designs that are vulnerable to fatigue cracking appear to include a) designs for large levels of traffic loading, specifically a TI of 15 (ESAL range $64.3 \times 10^6 – 84.7 \times 10^6$); b) designs for subgrades of intermediate strength, specifically an R-value of 20; c) designs for coastal regions, specifically Santa Barbara; and d) designs incorporating class B cement-treated base.
- Rich-bottom designs, in which the bottom portion of the asphalt surface course is enriched with added asphalt and compacted with fewer air voids, have the potential for enhancing fatigue performance, extending fatigue life by at least threefold beyond that of conventional structures in the absence of other adverse materials, construction, and traffic influences. The investigation of rich-bottom pavements, the results of which are shown in Figure 5, is an example of the power of the mix design and analysis system to quantitatively explore materials and structural alternatives that take advantage of improved fatigue properties at critical locations in the pavement structure.

2. *Accelerated HVS Loading Tests on Four Full-Scale Pavements (3,4,5,6,7,8)*.
Initial test plans for HVS tests include two objectives:

(1) to develop data to verify existing Caltrans design methodologies for drained, asphalt-treated permeable base (ATPB) pavements and conventional aggregate base (AB) pavements with regard to failure under trafficking at moderate temperatures, and to create a uniform platform on which overlays will be constructed; and



a. conventional pavement



b. rich-bottom pavement

Figure 5. Effect of surface thickness and design reliability on in-situ traffic resistance (conventional and rich-bottom pavements)

(2) to compare the fatigue performance of structural overlays of hot asphalt-rubber, gap-graded (ARHM-GG) mixes using type 2 asphalt-rubber binder with that of conventional dense-graded asphalt concrete (DGAC) mixes.

The pavement sections constructed for Objective (1) are shown in Figure 2. Testing of the overlays to meet Objective (2) has not been completed at this time. Accordingly, fatigue performance of these overlays will not be reported herein.

Prior to the start of the fatigue study of the overlays, however, an additional objective was added to the initial test plans. This objective encompassed an evaluation of the rutting propensity of the initial mix and of the two overlay mixes subjected to a range in tire types and for two load conditions. The initial results of this study are included in Part 4 of this section.

HVS trafficking of the first ATPB section (500RF) commenced on May 3, 1995 and was completed on November 8, 1995. Trafficking of the first AB section (501RF) commenced on November 20, 1995 and was completed on February 26, 1996. Dates associated with the other two sections (ATPB-502CT and (AB-503RF) are included in Table 3. A total of 2.57×10^6 load repetitions were applied to Section 500RF consisting of 150,000 repetitions of a 40-kN (9000-lb) load, 50,000 repetitions of an 80-kN (18,000-lb) load, and the remainder with a 100-kN (22,500-lb) load. The same initial loading sequence was applied to the other three sections. The total repetitions applied to each section and the repetitions at which the first load-associated cracks were observed are summarized in Table 3.

Table 3 Summary of results of first four HVS tests at RFS

Section	Base type	Loading period	Repetitions at which first load-associated cracks were observed	Total HVS repetitions to failure	Estimated ESALs[a]	Crack length/ Area (m/m^2)
500RF	ATPB	5/3/95–11/8/95	6.5×10^5	2.57×10^6	112×10^6	2.5
501RF	AB	11/20/95–2/26/96	5.5×10^5	1.43×10^6	59×10^6	9.6
502CT[b]	ATPB	12/5/95–9/20/96	1.3×10^6	2.67×10^6	117×10^6	4.0
503RF	AB	3/6/96–9/18/96	4.0×10^5	1.91×10^6	81×10^6	6.5

[a] Estimated according to the Caltrans relationship for load equivalency:

$$ESALs = \left[\left(\frac{actual\ load}{18{,}000} \right)^{4.2} \right]$$

[b] Trafficked with HVS No. 2.

At the termination of loading in each of the sections, the alligator-type cracking had reached a level which, according to Caltrans pavement management criteria, constituted fatigue failure. The density of cracking in the sections with untreated AB (undrained) was larger than that in the sections with ATPB (drained) as seen in Table 3.

To analyze the response of the four test sections, the fatigue analysis and design system described in the previous section was utilized. The program CIRCLY *(15)* was selected for the multilayer elastic analysis since it has provisions for both full friction and no friction between the interface of the AC layers.

In Table 3 it will be noted that there is a considerable difference between the loading tolerated by the test sections versus the design level of 1×10^6 ESALs. To explain this difference, a number of factors must be considered. Recognized and emphasized at the outset is the unknown error that results from assuming that the Caltrans design estimate is dictated primarily by the prevention of premature fatigue cracking and not other distress modes. Historically this design strategy has been successful.

A fundamental difference between a pavement design estimate and the corresponding test measurement is that design must accommodate a range of mixtures of varying performance characteristics and must incorporate a safety factor to prevent premature failure as a result of testing, construction, and traffic variabilities. A direct measurement in the HVS test, on the other hand, reflects the performance characteristics of a specific mix, it is independent of laboratory testing and traffic variabilities, and, because the test section is limited to a small area, 1 by 8 m in dimension, the influence of construction variabilities is minimal. The design estimate is expected to be smaller in magnitude than the test measurement with the difference between the two increasing as the design reliability increases. The fatigue design and analysis system has the capacity to distinguish between design estimates and test measurements and to assess the impact of reliability on design estimates. No reliability factor is associated with simulation of HVS test ESALs for the reasons described above.

Another difference between design estimates and test measurements relates primarily to pavement structural effects. For the pavement sections, notable differences exist between the design scenario including: 1) the thicknesses of the various layers (see Table 2); 2) a distinct difference in the air void contents within the two asphalt concrete lifts instead of uniform air voids; and 3) and an unbonded or weakly bonded interface between the lifts instead of a fully bonded one.

Table 4 Air-void contents of mix in HVS test sections

Asphalt Concrete Lift	Air-void content			
	500RF	501RF	502CT	503RF
Upper	7.8	7.2	4.1	4.8
Lower	4.4	5.6	2.4	4.4

Important differences are also found between the standard mix used for calibrating the fatigue analysis and design system and that used in HVS test sections. Asphalt and air-voids contents for the standard mix were set at 5 and 8 percent, respectively, representing the approximate asphalt content found from the Hveem mix design procedure and a relative compaction of about 97.3 percent. The asphalt content of the mix for the test sections averaged 4.8 percent, which was determined by the Hveem procedure. The air-voids contents varied in each section as shown in Table 4. Other differences existed between the standard mix and the mix used in the HVS test section as well.

To investigate these differences, five different cases were analyzed for each test section using the multi-layered elastic program CIRCLY, as noted above, to calculate a simulated response under traffic load. For test section 500RF, the cases were as follows:

Case	Description	Critical strain location
1	Design conditions with standard mix (8% air-voids content and 5% asphalt content); full friction interface	Bottom of lower lift
2	HVS conditions with standard mix (8% air-voids content and 5% asphalt content); full friction interface	Bottom of lower lift
3	HVS conditions with standard mix except for reduced air void content (4.4% air-voids content bottom lift, 7.8% air-voids content top lift and 5% asphalt content); full friction interface	Bottom of lower lift
4	HVS conditions with HVS mix (4.4% air-voids content bottom lift, 7.8% air-voids content top lift and 4.8% asphalt content); full friction interface	Bottom of lower lift
5	HVS conditions with HVS mix (4.4% air-voids content bottom lift, 7.8% air-voids content top lift and 4.8% asphalt content); frictionless interface	Bottom of upper lift

A distinction was made between design conditions and as-built and as-tested HVS conditions with respect to layer thicknesses (Table 2) and environmental influences. This latter influence was taken into account by assuming the moduli of the supporting layers was 80 percent of the HVS moduli. Elastic parameters used for the CIRCLY analyses are summarized in Table 5. For the asphalt concrete, moduli are based on stiffness measurements at 20 C. The modulus values for the ATPB and untreated materials are based on laboratory resilient modulus tests and results obtained by back calculation procedures from falling weight deflectometer (FWD) tests.

The first matter to be addressed includes both the effect of reliability on estimates of design ESALs as well as the fundamental difference between design ESALs, whether from the Caltrans or UCB procedures, and HVS ESALs whether measured under HVS loading or simulated using the UCB system. In computing HVS ESALs, variances of the several parameters (asphalt content, air-voids content, asphalt concrete thickness, foundation support, and traffic) were assumed to be negligible. Computations using the UCB system for Case 1 conditions yielded the ESAL estimates shown in Table 6; (i.e. simulated HVS ESALs=8.06×10^6 and ESALs for a reliability of 90 percent=2.16×10^6) demonstrate clearly the influence of reliability on design ESALs. It is interesting that the fatigue analysis and design system estimated 2.16×10^6 design ESALs at 90-

percent reliability, about twice the Caltrans design estimate of approximately 1×10^6 ESALs. Although not shown in Table 6, the fatigue analysis and design system estimated 0.98×10^6 ESALs at 98-percent reliability, very nearly the same as the Caltrans design estimate. As expected, the 8.17×10^6 simulated HVS ESALs is considerably greater than any of the design ESALs; for a design reliability level of 90 percent, the computed ratio of simulated HVS ESALs to UCB system design ESALs is approximately 3.7.

Next, in order to demonstrate the significant difference between as-built and as-tested HVS conditions and assumed Caltrans design conditions, the simulated HVS ESALs estimate for Case 2 was compared with that for Case 1, Table 6. The ratio of HVS ESALs for HVS conditions to that for design conditions is approximately 2.2. The enhanced simulated performance for the HVS environment stems from a combination of favorable thickness differences and environmental effects, including both temperature and moisture.

To show the effect of the excellent mix compaction achieved in the construction of Section 500RF, particularly the lower lift, the simulated HVS ESALs estimate for Case 3 was compared with that for Case 2, Table 6. The significant effect of air-void content is illustrated by a ratio of approximately 2.9 in the HVS ESALs estimate for a 4.4-percent bottom lift and 7.8-percent top lift compared to that for an 8-percent mix in both lifts. This finding emphasizes the benefits of good construction practice and greater compaction levels than current Caltrans specifications typically require. An even larger improvement in simulated fatigue life than that shown here would result if the 4.4-percent air-void content exists in both lifts. These corroborate similar findings from previous fatigue studies for a variety of typical Caltrans pavement structures *(1,2)*.

Table 5 Elastic parameters for CIRCLY analyses, Section 500RF

Layer	Case					All Cases
	1	2	3	4	5	
	Modulus (MPa)					Poisson's Ratio
Upper asphalt concrete lift	6736	6736	6839	7317	7317	0.35
Lower asphalt concrete lift	6736	6736	8848	9766	9766	0.35
ATPB	827	1034	1034	1034	1034	0.40
Aggregate base	240	300	300	300	300	0.35
Aggregate subbase	120	150	150	150	150	0.35
Subgrade	56	70	70	70	70	0.45

Table 6 Comparisons of measured and estimated ESALs, all test sections

	ESALs × 10^6							
	Simulated HVS ESALs				UCB System Design ESALs; Reliability = 90 percent			
	500RF	501RF	502CT	503RF	500RF	501RF	502CT	503RF
Case:								
1	8.06	2.45	8.06	2.45	2.16	0.656	2.16	0.656
2	18.1	4.76	19.4	8.63	4.73	1.25	5.08	2.19
3	53.0	11.7	83.6	36.2	13.8	3.08	21.9	9.20
4	292	66.0	456	216	76.2	17.3	119	55.0
5	6.74	6.42	9.68	17.1	2.10	2.05	3.05	5.22
Actual	112	59.0	117	81.0				

To demonstrate the effect of the superior fatigue performance of the HVS mix compared to the standard mix, the simulated HVS ESALs estimate for Case 4 was compared with that for Case 3, Table 6. The ratio of simulated HVS ESALs for these two conditions is approximately 5.5. This large difference is not clearly attributable to any one particular component of the two mixes and is likely due to a combination of potential differences in components including aggregate type, asphalt production, and aggregate gradation.

One such factor relates to the lack of bonding at the interface between upper and lower asphalt concrete lifts. CIRCLY enables analysis of two interface extremes: frictionless and full-friction. The interface condition of Section 500RF is probably somewhere between these two extremes. That is, the interface is rough (but unbonded) and the weight of the upper layer combined with vertical compressive stress beneath the load should allow some of the horizontal interface movement to be transmitted from one lift to the other. Even through partial friction conditions cannot be modeled with available techniques, the notable effect of interface condition can be demonstrated by comparing the HVS ESALs estimate for Case 5 with that for Case 4, which are shown in Table 6. This remarkably large effect is due to two factors, one of which is the nature of friction at the interface between lifts. The other results from a shift in the critical strain (i.e., initial crack) location from the bottom of the lower lift for the full friction interface to the bottom of the upper lift for the frictionless interface. In the upper lift, the air-void content is much greater, and load repetitions necessary to propagate cracks to the top surface are expected to be smaller because of the reduced thickness through which the cracks must propagate. The larger air-void content and reduced overlying thickness significantly decrease the simulated fatigue life.

Although this analysis is not definitive because the interface condition cannot be accurately modeled by CIRCLY, the estimate of 292 × 10^6 ESALs is reasonably comparable to HVS loading of 112 × 10^6 ESALs. The simulation results indicating that cracking would occur in the upper lift before it occurred in the lower lift for the frictionless interface condition corroborate the cracking observed in from Section 500RF cores.

The analysis reported herein provides explanations and insights in reconciling the difference between the Caltrans design estimate of 1×10^6 ESALs and the HVS test measurement of 112×10^6 ESALs. In addition, it has highlighted factors such as mix, air-void content, and interface condition and yielded quantitative estimates of their impact on pavement performance.

Results of HVS tests in Section 501RF, 502CT, and 503RF, (summarized in Table 6) support the above discussion. Moreover, as seen in Table 6, the design thickness of AC for the undrained section appears inadequate at a reliability level of 90 percent (ESALs less than 1×10^6), reinforcing the discussion presented earlier regarding the potential for fatigue cracking in Caltrans designed pavements for asphalt concrete on untreated aggregate base.

3. *Asphalt Treated Permeable Base (ATPB) Study (9).*

This study has included:

- Evaluation of the performance of asphalt treated permeable base (ATPB) in asphalt concrete pavements including: 1) a summary of Caltrans experience with ATPB and drainage systems, their development and implementation, and observations of field performance with respect to stripping and maintainability; and 2) a summary of the characteristics and performance of ATPB materials and drainage systems used by two other highway agencies.
- Laboratory investigation of the stiffness and permanent deformation characteristics of ATPB mixes including the effects of soaking and repeated loading while saturated.
- Analyses of representative pavement structures to determine the effects of both as-compacted and soaked ATPB on performance; the purpose of this latter study being to extend the applicability of the HVS tests results from Sections 500 and 502 to representative in-service conditions.

From the survey among the Caltrans Districts, some problems have been reported with the use of ATPB. Stripping of asphalt from the aggregate has been observed in some ATPB materials; this phenomenon has also been reported by other agencies using similar materials. Maintenance of edge drains has been a problem for some Caltrans districts, particularly where the drains have been added as retrofits rather than as design features in new or reconstructed pavements. In addition, several districts have reported frequent clogging of their drainage systems. These observations stress the importance of examining ATPB in a soaked state.

While the extent to which an ATPB might remain saturated has not been extensively examined, results of the study in Indiana *(9)* suggests that the ATPB can remain saturated for a substantial period of time after a rain event. These results indicate that it is important to study the response of saturated ATPB and that loading representative of moving traffic be applied to the material in this condition.

Results of the study show that, in addition to any benefits provided by improved pavement drainage, inclusion of an ATPB layer can increase the structural capacity of AC pavements with respect to fatigue cracking and subgrade rutting. However, better strucural capacity requires that the ATPB is resistant to stripping, loss of cohesion, and stiffness reduction from water damage. For these conditions the ATPB gravel factor can be increased to a value of the order of 2.0 from its current value of 1.4. If, on the other hand, water damage causes reduced stiffness, a gravel factor in the range 1.4 to 1.7 appears reasonable depending on the degree of water damage anticipated.

Current design criteria leave ATPB layers susceptible to substantial reduction in stiffness from the effects of water because of the low-specified asphalt content and lack of a water sensitivity evaluation. It is very likely that the performance of ATPB in the presence of water can be substantially improved through improved mix design, drainage design, construction procedures, and maintenance practices.

This study also recommends that Caltrans reconsider its use of ATPB directly under the AC layer to intercept water entering through the pavement surface. By reducing the permeability of the AC through improved compaction and incorporating sufficient thickness to mitigate the potential for load associated cracking, the need for the ATPB in this location could be eliminated.

While the stiffness values of ATPB in both the dry and wet state appear larger than representative stiffness values for untreated granular bases, they are at least an order of magnitude less than the stiffness of conventional AC. Accordingly, improved pavement performance could, from a cost standpoint, be better achieved by proper mix design and thickness selection [e.g., *use of the rich bottom concept (1)*] and through improved construction practices, particularly AC compaction.

4. Mix Permanent Deformation Studies.

In the period May to September 1997, HVS testing was performed with various wheel and tire types on the surface of the AC used for Sections 500-503 as well as on the surfaces of the overlays of these sections. Table 7 provides a summary of the tires used and of the 10 tests conducted. For all of the tests, the HVS load carriage was operated in the channelized mode, i.e., without wander. By operating the HVS in this mode, the resulting data can be utilized to validate analytical-based models that predict permanent deformation in the field from mechanical properties of mixes measured in the laboratory. In addition, it provides the opportunity to evaluate SHRP developed technology for mix design to mitigate permanent deformation, e.g., use of the simple shear test *(16)*.

Figure 6 illustrates the development of rut depth with load applications for tests conducted with the pavement temperature at a depth of 50 mm equal to 50 C. It will be noted that in this instance, the dual wheels with bias-ply and radial-ply tires produce the same amount of rutting; the wide-base single tire results in about a 25 percent increase in rutting over the dual configuration; and the aircraft wheel causes significantly more rutting than the other tires for these conditions. Figure 7 provides a summary of the repetitions to a rut depth of 13 mm for the 10 tests.

Predictions of permanent deformation using results of the simple shear tests performed on cores of the representative AC mixes have not yet been completed. The HVS test results do, however, generally conform to expectations of rutting and are reasonable for the different conditions evaluated.

These 10 tests were completed in a total of about 14 days of loading. The results provide an indication of the usefulness of the HVS test to quickly examine rutting conditions and serve therefore as a valuable validation technique for evaluation of improved mix analysis and design methods used to evaluate this mode of distress.

Table 7 Experiment matrix for CAL/APT HVS No. 1 tests for rutting of the original overlay mixes

Section	Mix type	Wheel type	Loading condition	Speed (std dev)	Temp @ 50 mm
504RF	Original DGAC (137 mm)	wide base single	40 kN, 110psi	7.69 km/h (0.35)	50 C
505RF	Overlay DGAC (75 mm)	bias-ply dual	40 kN, 100psi	7.74 km/h (0.54)	50 C
506RF	Overlay DGAC (75 mm)	radial dual	40 kN, 105psi	6.94 km/h (0.25)	50 C
507RF	Overlay DGAC (75 mm)	wide base single	40 kN, 110psi	7.69 km/h (0.35)	50 C
508RF	Overlay ARHM thick (60 mm)	wide base single	40 kN, 110psi	[a]	50 C
509RF	Overlay ARHM thick (60 mm)	radial dual	40 kN, 105psi	[a]	50 C
510RF	Overlay ARHM thin (37 mm)	radial dual	40 kN, 105psi	[a]	50 C
511RF	Overlay ARHM thin (37 mm)	wide base single	40 kN, 110psi	[a]	50 C
512RF	Overlay DGAC (75 mm)	wide base single	40 kN, 110psi	[a]	40 C
513RF	Overlay DGAC (75 mm)	aircraft wheel	100 kN, 150psi	[a]	50 C

- Bias-ply dual, 10.00H20 inches, tread 6 plies nylon cord, sidewalls 6 plies nylon cord, maximum dual load 6,300 lbs at 90 psi cold
- Radial-ply dual, 11R22.5, tread 6 plies steel cord, sidewalls 1 plies steel cord, maximum dual load 5,750 lbs at 105 psi cold
- Wide base single, 425/65R22.5, tread 5 plies steel cord, sidewalls 1 ply steel cord, maximum load 10,500 lbs at 110 psi cold
- Aircraft wheel, 46H15, reinforced thread, tubeless, 44,800 lbs at rated pressure

[a] The wheel speeds were relatively constant for the three wheel types so they were not measured in subsequent tests.

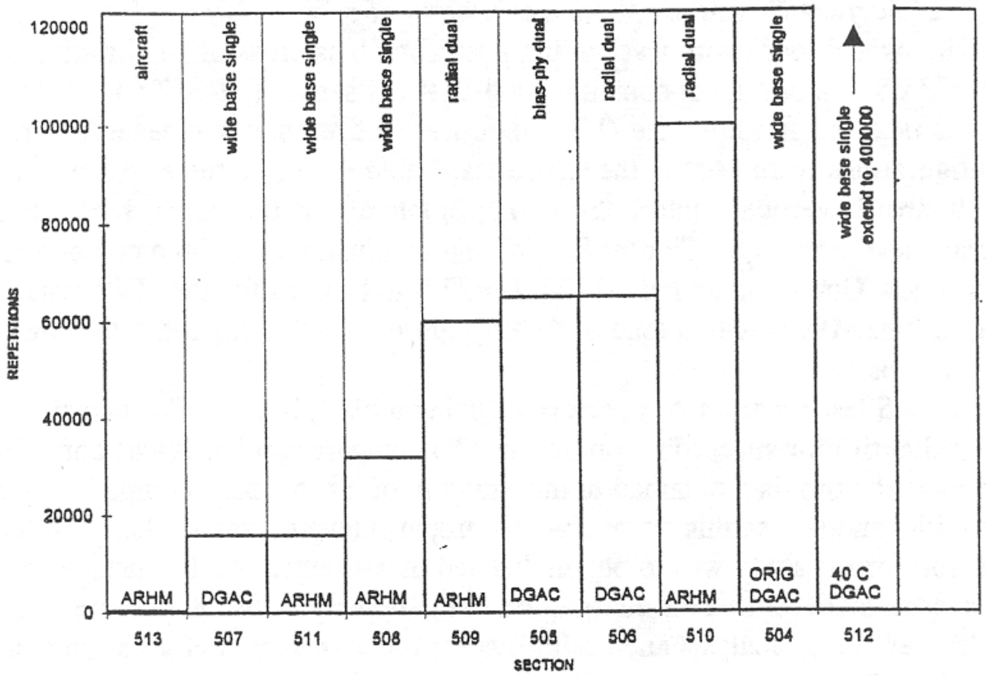


Figure 6 Average maximum rut, influence of wheel type on DGAC

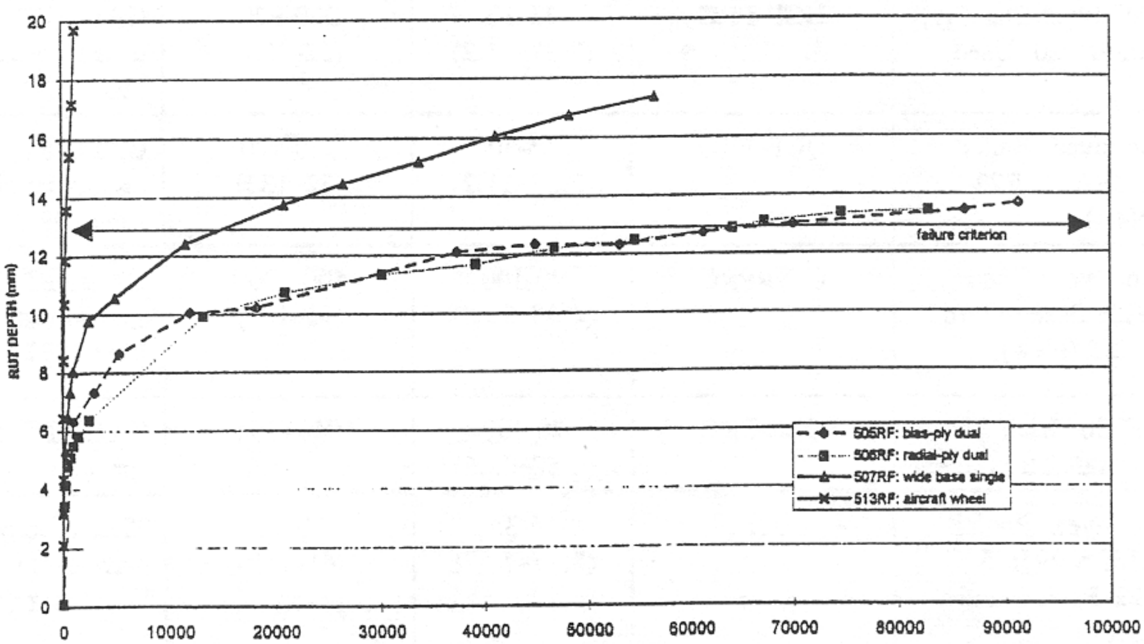


Figure 7 Summary of the repetitions to a rut depth of 13 mm for the ten tests

5. Three-Dimensional (3-D) Contact Stresses Between Tire and Pavement Surface Measured by Vehicle-Road Surface Pressure Transducer Array (VRSPTA) (10).

Slow moving wheel loads with free-rolling pneumatic bias/cross ply and radial tires were applied by the HVS to a pavement containing a 3-D stress sensor (VRSPTA). A description of the VRSPTA is included in Reference *(10)*. Six different tire types and several load/inflation pressure configurations were used in the test series, Table 8. The stresses that were measured simultaneously are: 1) vertical contact stress, σ_{zz}; 2) lateral (or transverse) shear stress, τ_{zy}; and 3) longitudinal shear stress τ_{zx}. Figures 8a, 8b, and 8c illustrate the three components of stress measured for a new Goodyear Radial G159A 11R22.5 to be used for the HVS tests commencing in June, 1997, with a load of 41 kN (approx. 20,000 lb) and a tire pressure of 720 kPa (approx. 105 psi).

The initial HVS test program was performed using a bias ply tire. The decision was made to change to radial tire for subsequent programs. To compare results, it was considered important to have the tire data obtained in this study in order to insure compatibility of test data between the initial and subsequent programs and proper interpretation of their results. Another important reason for the study was to obtain detailed measurement of the surface contact stresses in order to model the stress distribution in the upper part of the pavement to insure the proper development of an analysis and design system for permanent deformation (rutting) estimation in the AC layer.

Table 8 Tires tested in VRSPTA study

Tire Type	**Organization Using Tire**	**Tire Loads kN (lbs × 10^3)**	**Tire Pressures kPa (psi)**	**Tire Use**
Goodyear Bias Ply, 10.00 × 20 (Used)	UCB-PRC	15-50 (3.37–11.2)	220-920 (32–133)	CAL/APT, HVS-1 tests through May 1997
Goodyear Radial G159A, 11R22.5 (New)	UCB-PRC	15-50 (3.37–11.2)	220-920 (32–133)	CAL/APT, HVS-1 tests beginning June 1997
Goodyear G286, Wide Base, 425/65 R22.5 (New)	UCB-PRC	20-100 (4.50–22.5)	500-1000 (73–145)	CAL/APT, HVS-1 special tests after June 1997
BF Goodrich Aircraft Tire (Used)	UCB-PRC	20-100 (4.50–22.5)	1040 (151)	CAL/APT, HVS-1 special tests
Goodyear Radial G159A 295/75 R22.5 (New and Used)[a]	NATC	15-35 (3.37–7.87)	420-820 (61–119)	Tires used on driverless trucks at WesTrack ()
Goodyear G178 Wide Base, 385/65 R22.5 (New)	NATC	30-50 (6.74–11.2)	500-1000 (73–145)	Typical off-road application tire

[a]New tires tested after 100 mile “run-in” at WesTrack; operating velocity—40mph

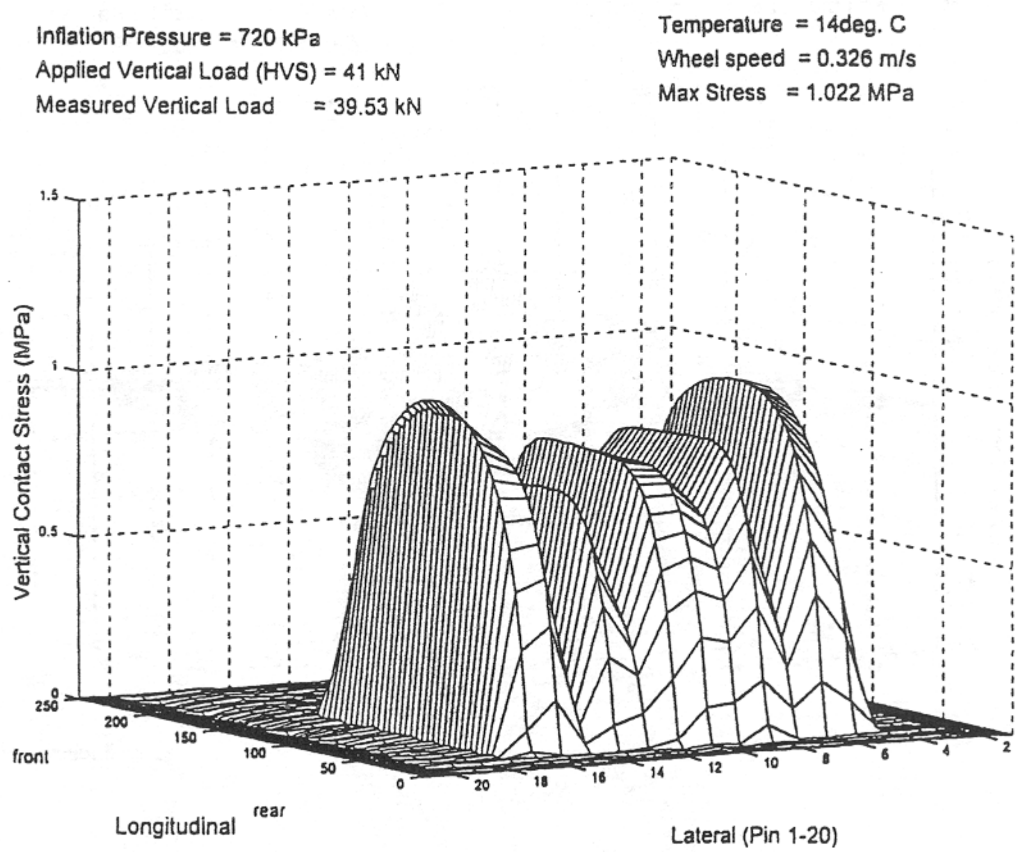


Figure 8a Vertical contact stress, σ_{zz}, distributions, Goodyear radial tire 11R22.5; inflation pressure—720kPa; vertical load—41kN

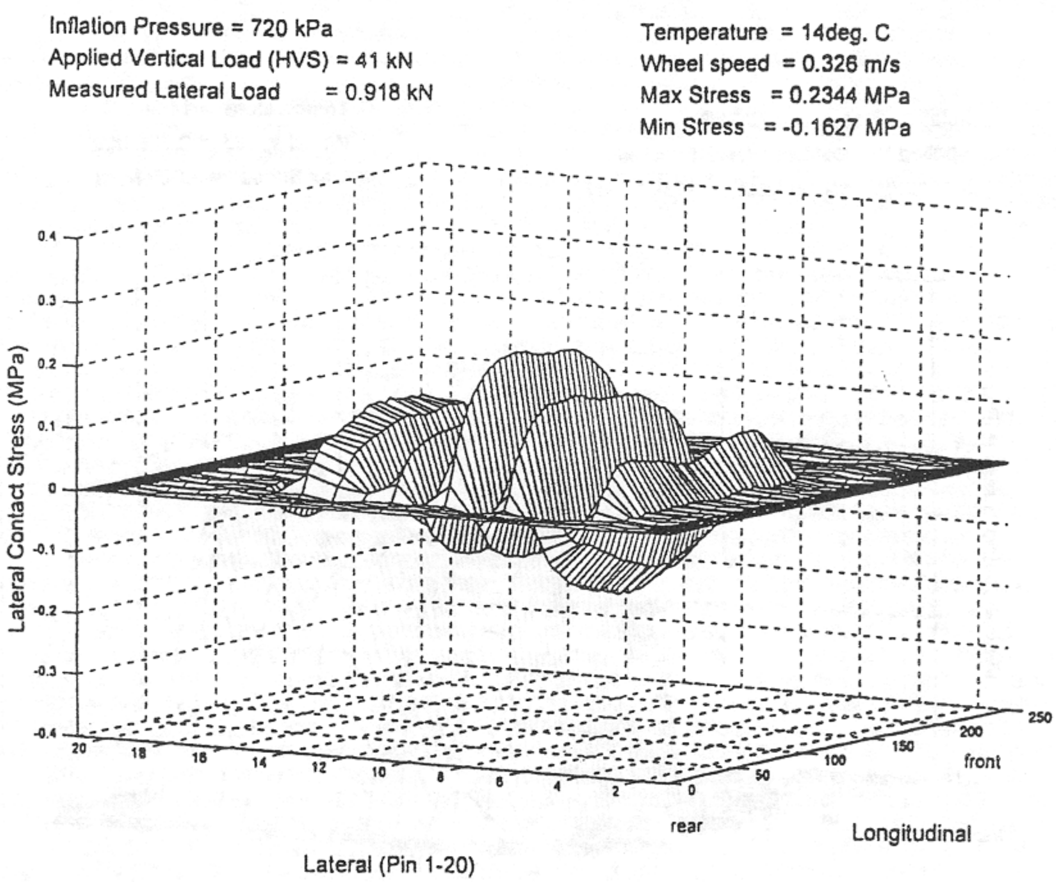


Figure 8b Lateral contact stress, τ_{zy}, distributions, Goodyear radial tire 11R22.5; inflation pressure—720kPa; vertical load—41kN

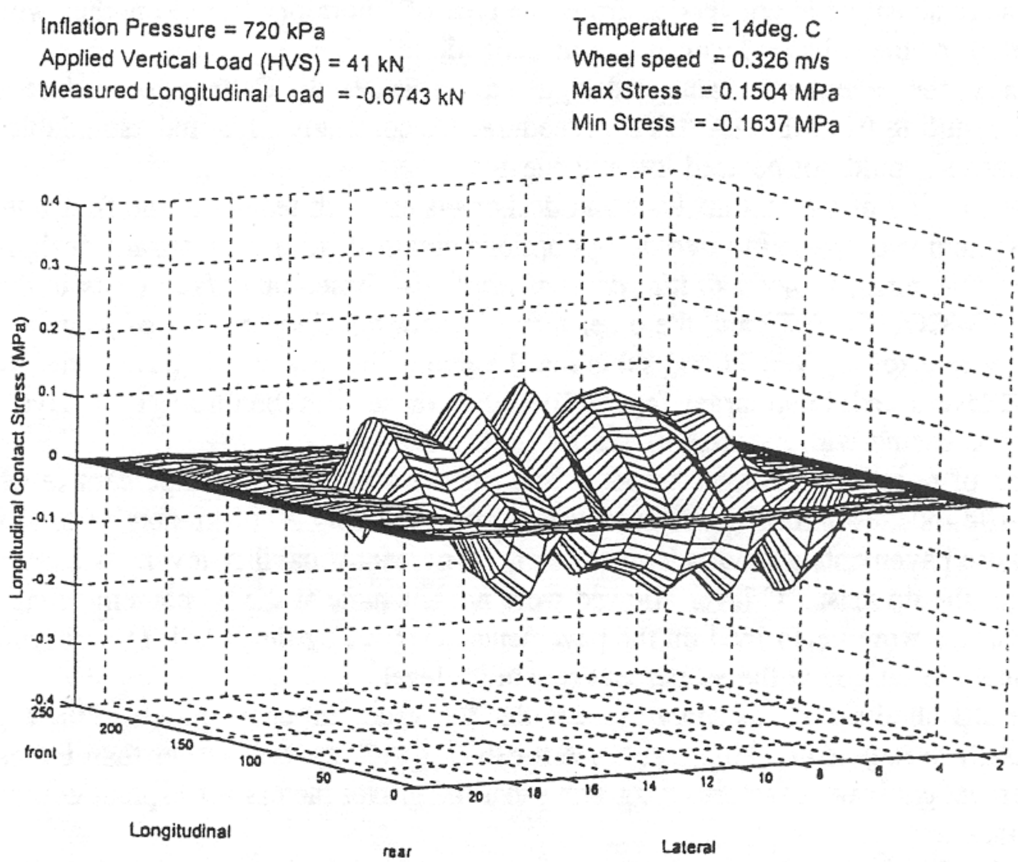


Figure 8c Longitudinal contact stress, τ_{zx}, distributions, Goodyear radial tire 11R22.5; inflation pressure—720kPa; vertical load—41kN

6. Comparison of the AASHTO and Caltrans Pavement Thickness Design Procedures (11).

While this study's primary objective was to compare thicknesses determined by the Caltrans and AASHTO procedures for a range in traffic conditions and subgrade strengths, it also included evaluations of: 1) the performance of pavements designed by the Caltrans procedure with equal gravel equivalents but with different thicknesses of asphalt concrete; and 2) the effects of different drainage conditions on pavement thicknesses according to the AASHTO procedure together with comparisons of these thicknesses with those obtained by the Caltrans method for the same traffic and subgrade conditions. The initial comparisons included pavement structures subjected to a range in traffic, as represented by Traffic Indexes of 7, 9, 11, 13, and 15, and a range in subgrade strengths, as measured by subgrade R-values of 5, 20, and 40.

The analyses indicate that, for the same inputs, the AASHTO and Caltrans pavement thickness design procedures do not produce the same pavement thicknesses. This is based on currently used conversions of subgrade material response characteristics since the two procedures utilize different measures of subgrade response: Caltrans using the R-value and AASHTO the resilient modulus, M_R. However, it should be noted that the two design procedures are sensitive to conversion from one type of laboratory test to another, which was demonstrated for the subbase layer in this investigation.

Generally, the pavement structures designed according to the Caltrans procedure are thicker than those resulting from the AASHTO procedure. Accordingly, it is understandable that the two procedures should not be used interchangeably.

After validating of the mix analysis and design system with results of the first fours HVS tests, the system was applied to two design options obtained from the Caltrans design procedure. One termed *lowest (initial construction) cost* [based on default costs in the computer program NEWCON 90 *(17)*] and the other termed *thinnest AC layer allowed* provided the same gravel equivalent for a given TI and subgrade R value. The lowest cost pavements, which have thicker AC layers, exhibited larger fatigue lives than those with the thinnest AC layer, the difference increasing with increase in Traffic Index, Table 9.

In terms of reliability the lowest cost pavements are adequate at a 90 percent reliability level for all Traffic Indexes and subgrade R values of approximately 20 or greater. For R-values less than 20, these pavements are not adequate at the 90 percent reliability level. All Caltrans designs with the thinnest AC layer allowed were not adequate at the 90 percent reliability level. Similar analyses were performed on the pavements designed by the AASHTO procedure. None were adequate in fatigue at the 90 percent reliability level.

The results obtained in this study indicate that the structural contribution of the asphalt concrete (AC) to fatigue cracking resistance for thicker AC layers is larger than indicated by the Caltrans gravel equivalent factors, suggesting that the gravel factors for asphalt concrete should be re-evaluated.

All of the above analyses were made using a conventional mix containing a California Valley asphalt. Improved performance was obtained using a California Coastal asphalt. This improved performance did not, however, change the above conclusions regarding fatigue response.

Table 9 Ratio of predicted fatigue to design ESALs, 90 percent reliability level

Traffic Index	TI Caltrans ESALs	Subgrade R-Value	Caltrans Lowest Cost	Caltrans Thinnest AC	AASHTO
7	120,000	5	0.97	0.44	0.28
7	120,000	20	0.85	0.54	0.33
7	120,000	40	1.32	0.77	0.57
9	1,000,000	5	0.29	0.29	0.12
9	1,000,000	20	1.38	0.37	0.13
9	1,000,000	40	2.17	0.53	0.23
11	5,400,000	5	0.64	0.17	0.09
11	5,400,000	20	2.18	0.20	0.11
11	5,400,000	40	2.25	0.28	0.16
13	22,000,000	5	0.59	0.09	0.10
13	22,000,000	20	1.27	0.11	0.13
13	22,000,000	40	2.87	0.14	0.20
15	73,160,000	5	0.81	0.07	0.12
15	73,160,000	20	1.70	0.09	0.15
15	73,160,000	40	3.72	0.11	0.23

The AASHTO procedure includes provision for different drainage conditions. While the thickness of the AC remains constant, thicknesses of base and subbase can change substantially depending on anticipated drainage conditions. On the other hand, the Caltrans procedure assumes that pavement designs without special drainage provisions are adequate and that their inclusion make Caltrans pavement designs conservative, but the extent of this conservatism is not known, which mechanistic-empirical methods can help quantify.

The investigation has demonstrated the ability of the mechanistic-empirical pavement analysis and design procedure to quantitatively evaluate the effects of pavement structure, materials selection, and subgrade strength on a specific mode of pavement distress. In addition, it has demonstrated that the influence of drainage conditions on pavement performance can be directly investigated provided measured stiffness characteristics of the unbound materials for a range in degrees of saturation are available. The evidence provided by this study as well as other investigations in the program strongly suggest that Caltrans should consider moving towards implementation of a mechanistic-empirical fatigue analysis and design procedure for the design of asphalt concrete pavements.

7. Pay Factor Study (12).

With Caltrans instituting a QC/QA system for asphalt concrete construction and establishing penalties/bonuses (pay factors) related to the quality of construction, results from the CAL/APT program coupled with earlier SHRPCdeveloped information on mix performances have provided a rational and feasible method for establishing these penalties and bonuses.

The approach taken focused principally on economic impacts to a highway agency (in this case, Caltrans). It assumed that an appropriate penalty for inferior construction should be the added cost to the agency. It also assumed that the bonus for superior construction should be no greater than the added savings to the agency.

For new construction, these agency costs/savings are associated primarily with subsequent pavement rehabilitation. Inferior construction hastens future rehabilitation and may increase the cost of rehabilitation as well. As a result, inferior construction increases the present worth of future rehabilitation costs. Superior construction, on the other hand, reduces the present worth of these costs largely by deferring the future rehabilitation. The difference in present worths of rehabilitation costs, as constructed versus as specified and as expected, provides a rational basis for setting the level of penalty/bonus for inferior/superior construction quality.

Computation of the differential present worth of future rehabilitation required a performance model for determining the effect of construction quality on anticipated pavement performance and a cost model for translating these effects into rehabilitation dollars. The performance model which has been used is based on a mix analysis and design system originally developed as a part of SHRP, extended to efficiently treat in-situ temperatures, calibrated to the Caltrans flexible-pavement design system, extended to incorporate construction variability, and most recently used in interpreting results of the Phase I Caltrans Heavy Vehicle Simulator (HVS) testing of the CAL/APT program. As applied thus far, it is limited to fatigue distress and specifically considers the means and variances of the following AC construction quantities; asphalt content, air-voids content, and AC thickness. In estimating damaging strains under traffic loading, it treats the pavement as a multilayer, elastic system. The performance model computes the distribution of pavement fatigue life, expressed as ESALs, using Monte Carlo simulation techniques.

The cost model which has been used is a simple one which considers only the time to the next rehabilitation activity. It understates agency costs by ignoring possible effects of construction quality on future rehabilitation costs; it also ignores future rehabilitation activity beyond the first cycle. It requires an estimate of future rehabilitation cost (in current-year dollars) and considers annual inflation of rehabilitation costs, traffic growth, expected years of new pavement life, and a discount rate representing the time value of money.

The framework for the performance model is outlined schematically in Figure 9. Air-void content, asphalt content, and AC thickness are selected randomly for each simulation. Although not shown in Figure 9, a random selection is also made of the foundation modulus, a modulus representing the composite effects of base, subbase, and subgrade layers in an "equivalent" two-layer elastic system.

These random selections assume normally distributed random variables with known or assumed means and variances. Of particular significance are the variances that might be expected under normal construction operations. Estimates of these variances were obtained from a combination of literature review, moduli backcalculations of FWD measurements, and unpublished data recently collected as part of the WesTrack project.

Table 10 summarizes the quantities used herein to represent reasonable estimates of materials/construction variability associated with conventional construction practice. The equations for estimating the standard deviation of asphalt-concrete thickness were developed herein as an approximate way to handle multi-lift construction. Among the assumptions made in their development was that the coefficient of variation of thickness in single-lift construction is about 14 percent *(12)*.

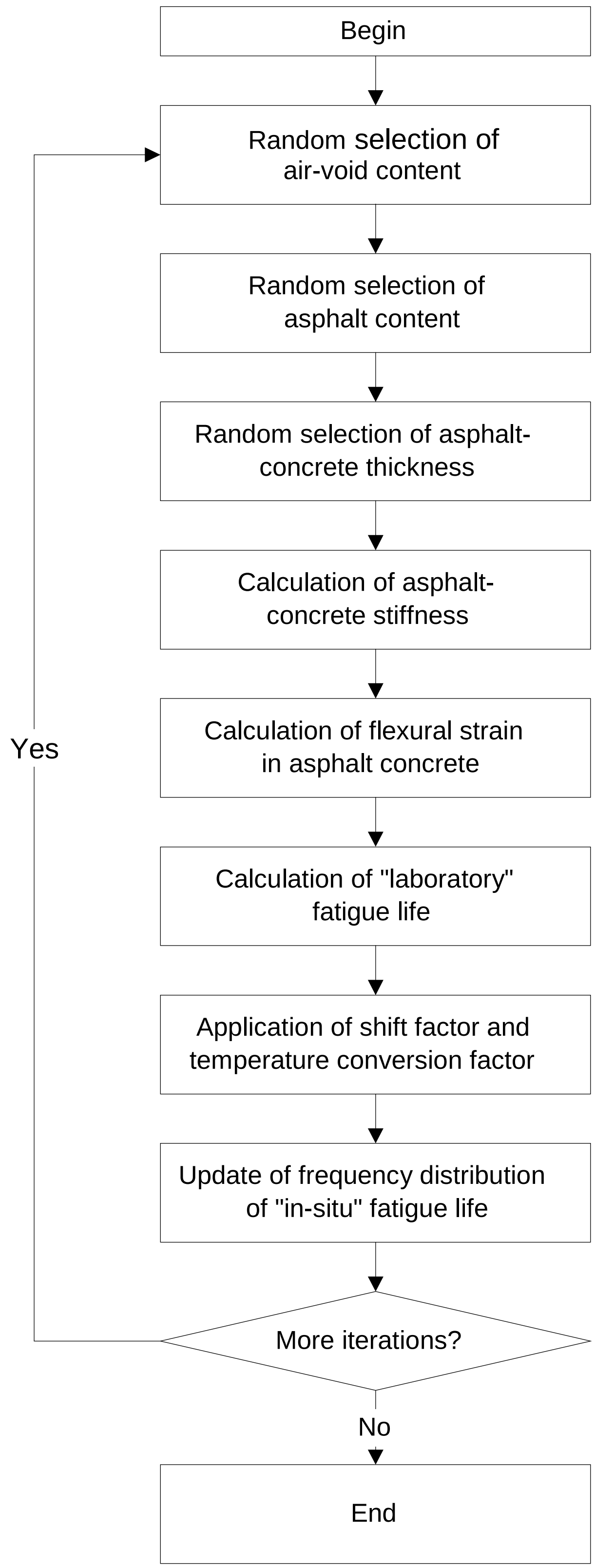


Figure 9 Outline of the pavement performance model simulation

Table 10 Variation of mix and structural characteristics for Monte Carlo simulations

Parameter	Total standard deviation	Percentage of variance due to materials/construction	Materials/construction component of standard deviation
Asphalt content	0.30%	40	0.19%
Air-voids content	1.6%	60	1.2%
Surface thickness	$0.200 \times T^{0.69}$(cm)	80	$0.173 \times T^{0.69}$(cm)
Foundation modulus	30% (coefficient of variation)	70	25%

T = thickness

The multilayer elastic program, ELSYM5, was used to simulate the stress and strain states within the simulated pavement structures. Loading consisted of a dual-tire assembly of 9,000 pounds (total) having a center-to-center spacing of 12 inches and a contact-pressure of 100 psi. The critically stressed location for fatigue was assumed to be at the bottom boundary of the asphalt-concrete layer.

The 10^{th}-percentile fatigue life was used as the basic performance estimate. This life corresponds to about 10-percent fatigue cracking in the wheel paths. A sensitivity analysis showed that incremental agency costs due to off-target construction (of either inferior or superior quality) are not significantly affected by the chosen performance percentile (at least within a reasonable range of the 1^{st} to the 20^{th} percentile).

The performance model which has been used yields the 10^{th}-percentile in-situ fatigue lives, including ESALs, for both expected or on-target construction quality and off-target construction quality. The relative performance, RP, the performance input to the cost model, was computed as follows:

$$RP = \frac{off\text{ - }targetESALs}{on\text{ - }targetESALs}$$

The off-target pavement life in years, OTY, that results from the simulated performance differential, assuming that traffic grows geometrically, was computed as follows:

$$OTY = \frac{\ln(1 + RP[(1 + g)^{TY} - 1])}{\ln(1 + g)}$$

in which g is the annual rate of traffic growth expressed as a decimal and TY is the number of years of pavement life resulting from on-target construction activity.

The cost model assesses the present worth of moving the first rehabilitation cycle from its on-target position, *TY*, to its off-target position, *OTY*. The net present worth, as expressed as a percentage of the rehabilitation costs (in current-year dollars) is computed as follows:

$$\Delta PW = 100\left(\frac{1+d}{1+r}\right)^{TY}\left(\frac{1+r}{1+d}\right)^{OTY} - 100$$

in which ΔPW is the percentage change in the present worth of the cost of the first rehabilitation cycle, r is the annual rate of construction-cost inflation expressed as a decimal, and d is the annual discount rate expressed as a decimal. Applying this percentage to the expected rehabilitation cost yields the agency cost increment due to off-target construction.

Using the approach described, combined pay factors were developed to consider the effects of relative compaction (air voids), asphalt content, and asphalt concrete thickness. Using a 95-percent relative compaction (and assuming an air-void content at maximum laboratory density of 5 percent), an air-void target of 7 percent was considered to be a reasonable expectation under current construction norms (a standard deviation in air-void content of 1.2 percent). Other reasonable targets for normal construction activity included a mean asphalt content equivalent to the job-mix formula with a standard deviation of 0.19 percent (Table 10) and a mean AC thickness equivalent to the design thickness with a standard deviation as determined by the equation in Table 10.

A set of simulations was then performed to quantify the effects of construction quality (air voids/relative compaction, asphalt content, and AC thickness) on both simulated in-situ fatigue performance as well as agency rehabilitation costs. Four pavement sections, typical of California construction over a range of Traffic Indexes (7 through 13), were evaluated. Each investigation employed either 100,000 simulations (air voids/relative compaction and asphalt-concrete thickness) or 200,000 simulations (asphalt content).

Rational pay factors calculated from these analyses were then combined to produce the combined pay factors shown in Table 11. These were determined from the following:

$$CPF = (1 + pf_{av})(1 + pf_{ac})(1 + pf_{t}) - 1$$

where pf_{av}, pf_{ac}, pf_t = pay factors for air-voids content, asphalt content, and asphalt concrete thickness, respectively. As seen in Table 11, 5 percent increments in each of the variables have been utilized.

Recommendations have been made to Caltrans about the use of pay factors or some variation thereof on a trial basis.

Table 11 Recommended combined contractor pay factors (Percentage of future rehabilitation cost in current-year dollars

Contractor pay factor for asphalt content (%)	Contractor pay factor for asphalt-concrete thickness (%)	Contractor pay factor for relative compaction (%)										
		15	10	5	0	-5	-10	-15	-20	-25	-30	-35
5	25	51	44	38	31	25	18	12	5	-2	-8	-15
5	20	45	39	32	26	20	13	7	1	-5	-12	-18
5	15	39	33	27	21	15	9	3	-3	-9	-15	-22
5	10	33	27	21	16	10	4	-2	-8	-13	-19	-25
5	5	27	21	16	10	5	-1	-6	-12	-17	-23	-28
5	0	21	16	10	5	0	-5	-11	-16	-21	-27	-32
5	-5	15	10	5	0	-5	-10	-15	-20	-25	-30	-35
5	-10	9	4	-1	-5	-10	-15	-20	-24	-29	-34	-39
5	-15	3	-2	-6	-11	-15	-20	-24	-29	-33	-38	-42
5	-20	-3	-8	-12	-16	-20	-24	-29	-33	-37	-41	-45
5	-25	-9	-13	-17	-21	-25	-29	-33	-37	-41	-45	-49
5	-30	-15	-19	-23	-27	-30	-34	-38	-41	-45	-49	-52
5	-35	-22	-25	-28	-32	-35	-39	-42	-45	-49	-52	-56
5	-40	-28	-31	-34	-37	-40	-43	-46	-50	-53	-56	-59
0	25	44	38	31	25	19	13	6	0	-6	-13	-19
0	20	38	32	26	20	14	8	2	-4	-10	-16	-22
0	15	32	27	21	15	9	3	-2	-8	-14	-20	-25
0	10	27	21	16	10	4	-1	-6	-12	-18	-23	-29
0	5	21	16	10	5	0	-5	-11	-16	-21	-27	-32
0	0	15	10	5	0	-5	-10	-15	-20	-25	-30	-35
0	-5	9	4	0	-5	-10	-15	-19	-24	-29	-34	-38
0	-10	3	-1	-5	-10	-15	-19	-24	-28	-33	-37	-42
0	-15	-2	-6	-11	-15	-19	-24	-28	-32	-36	-41	-45
0	-20	-8	-12	-16	-20	-24	-28	-32	-36	-40	-44	-48
0	-25	-14	-18	-21	-25	-29	-33	-36	-40	-44	-48	-51
0	-30	-20	-23	-27	-30	-34	-37	-41	-44	-48	-51	-55
0	-35	-25	-29	-32	-35	-38	-42	-45	-48	-51	-55	-58
0	-40	-31	-34	-37	-40	-43	-46	-49	-52	-55	-58	-61
-5	25	37	31	25	19	13	7	1	-5	-11	-17	-23
-5	20	31	25	20	14	8	3	-3	-9	-15	-20	-26
-5	15	26	20	15	9	4	-2	-7	-13	-18	-24	-29
-5	10	20	15	10	4	-1	-6	-11	-16	-22	-27	-32
-5	5	15	10	5	0	-5	-10	-15	-20	-25	-30	-35
-5	0	9	4	0	-5	-10	-15	-19	-24	-29	-34	-38
-5	-5	4	-1	-5	-10	-14	-19	-23	-28	-32	-37	-41
-5	-10	-2	-6	-10	-15	-19	-23	-27	-32	-36	-40	-44
-5	-15	-7	-11	-15	-19	-23	-27	-31	-35	-39	-43	-48
-5	-20	-13	-16	-20	-24	-28	-32	-35	-39	-43	-47	-51
-5	-25	-18	-22	-25	-29	-32	-36	-39	-43	-47	-50	-54
-5	-30	-24	-27	-30	-34	-37	-40	-43	-47	-50	-53	-57
-5	-35	-29	-32	-35	-38	-41	-44	-48	-51	-54	-57	-60
-5	-40	-34	-37	-40	-43	-46	-49	-52	-54	-57	-60	-63

Note: Bonuses are positive and penalties are negative

8. Effects of Binder Loss Stiffness (SHRP PG Binder Specification Requirement) on Fatigue Performance of Pavements (13).

This study evaluated the influence of binder loss stiffness ($G^* \sin\delta$), the parameter included in the SHRP binder specification to control fatigue response in pavement performance. Results from two studies were included, namely: 1) those performed during the SHRP research program, which evaluated the fatigue response of mixes containing 8 asphalts and two aggregates and the simulated performance in representative pavement structures *(14)*; and 2) a detailed study of the effects of binder loss stiffness on the simulated fatigue performance of 18 different pavement sections designed according to the Caltrans procedures and evaluated in three different temperature environments in California *(1)*. (N.B. These are the same pavement sections used for the analyses reported in the fatigue study and described earlier in Section 1).

Results of both investigations indicated that the loss stiffness of the binder is not by itself a sufficient indicator of the fatigue performance of asphalt concrete in pavement structures. Moreover, these studies have highlighted the basic difference between the current binder specification (which sets a maximum limit on loss modulus) and field performance simulations which suggest that larger moduli are beneficial for most pavement structures. Figure 10 illustrates this point for the 18 pavement sections designed by the Caltrans procedure.

This study emphasized that the binder alone does not determine fatigue response in the pavement structure. Mix characteristics as well as the pavement structure itself and the environment within which it is located have a significant role in determining pavement performance.

Synthesis of Findings

From July 1, 1994 through June 30, 1997, 8 studies have been completed within the framework of Phase II of the CAL/APT program. Results from each of these studies are summarized in the previous sections. When viewed collectively, a number of conclusions and recommendations emerge, which could result in a significant improvement in asphalt pavement performance.

The mix design and analysis system originally developed as part of SHRP has been extended to efficiently treat in-situ temperatures, calibrated to the Caltrans asphalt concrete pavement design system, extended to incorporate construction variability and used to interpret the results of the first four HVS tests completed at the Richmond Field Station.

With increased confidence resulting from the successful interpretation of the HVS test results, use of this system provides the basis for what we believe are sound recommendations to Caltrans on three issues:

- Method of asphalt concrete pavement design
- Design and use of asphalt treated permeable bases
- Construction practices.

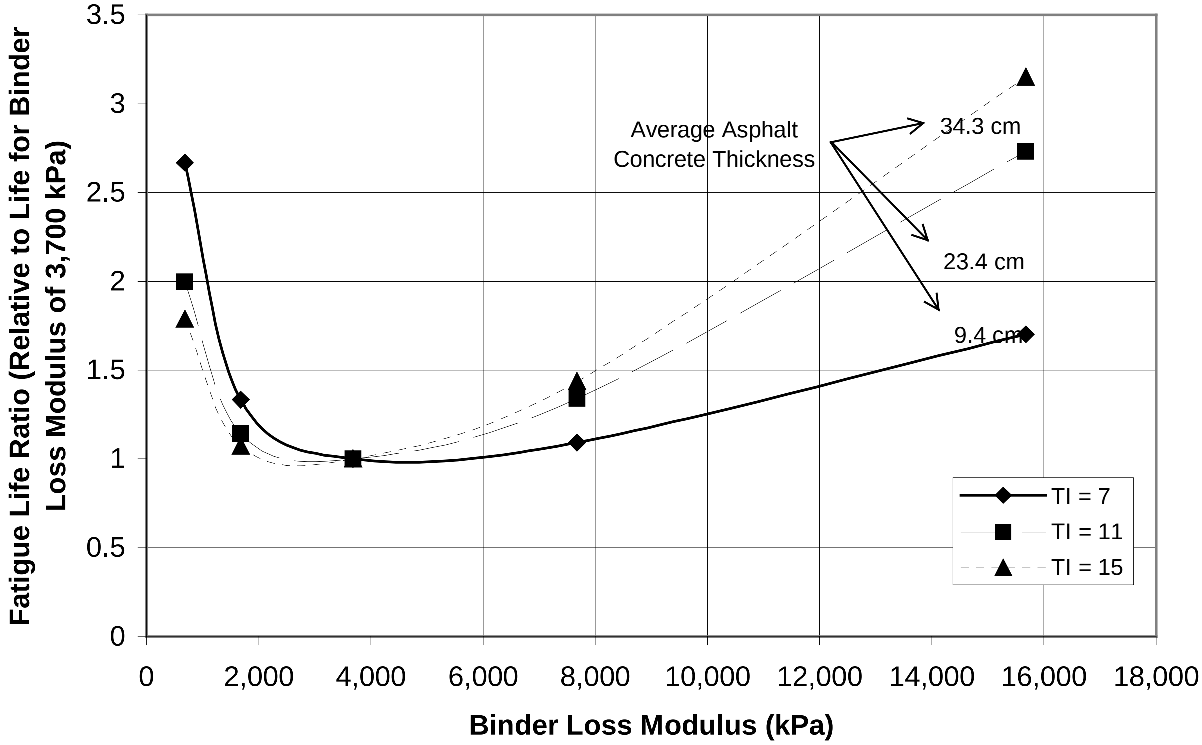


Figure 10 Relationship between average fatigue life ratio and binder loss modulus for 18 hypothetical pavement structures

Asphalt Concrete Pavement Design

Results indicate that the total pavement thicknesses developed by the current pavement design procedure are generally adequate to preclude the incidence of rutting resulting from permanent deformations occurring in the subgrade and untreated aggregate pavement layers. On the other hand, thicknesses of the asphalt concrete layers (particularly for weaker subgrades and higher traffic levels) probably are inadequate leading to premature fatigue cracking. Utilization of the mix design and analysis system described herein would reduce this propensity for fatigue cracking and generally improve the reliability of the design. Innovative pavement designs could extend fatigue lives substantially beyond conventional designs.

The mix design and analysis system would permit the more effective utilization of materials, e.g., the use of the "rich bottom" concept as well as insuring the effective use of new materials such as modified binders. Innovative pavement designs could extend fatigue lives substantially beyond conventional designs.

With the move of Caltrans toward the use of QC/QA procedures in pavement construction, this system provides a rational procedure in the development of pay factors. Using the calibrated mix analysis and design procedure, combined pay factors—including the effects of degree of mix compaction (as represented by relative compaction), asphalt content, and asphalt concrete thickness—have already been recommended.

Innovations in design and related improvements in construction quality will provide new asphalt concrete alternatives for longer life pavements in California.

Design and Use of Asphalt Treated Permeable Bases

From the HVS tests on the pavement sections containing ATPB, laboratory tests on representative ATPB mixes, and associated analyses, the general use of ATPB directly under the dense-graded asphalt concrete layer in the pavement section should be reconsidered. Improved compaction in the asphalt concrete layer will reduce its permeability. Increasing the thickness of the asphalt concrete, following the mix design and analysis system described herein will substantially delay crack initiation and propagation in this layer from repeated loading. Thus, by reducing the permeability and cracking potential of the asphalt concrete, the necessity for ATPB in this location could be substantially reduced or eliminated. It is likely that the resistance to cracking and reduction in permeability will be improved as well by the use of a "rich bottom" layer of asphalt concrete. Despite the steps to reduce the propensity for surface water to enter the pavement, it must be recognized that drainage layers may be required to help remove seepage entering the pavement structure through the subgrade.

If Caltrans continues to place ATPB directly beneath the asphalt concrete, then improvements could be made to the ATPB to enhance its performance. To enhance the performance characteristics of this material in the presence of water, an increase in binder content, the use of modified binders such as asphalt rubber, and the use of an additive such as lime or an anti-stripping agent should be considered. Associated with the changed mix design is the necessity for incorporation of properly designed soil or geotextile filters adjacent to the ATPB layer in the pavement structure to prevent it from clogging. Finally, to insure continued effectiveness of the ATPB, effective maintenance practices for edge and transverse drains should be established.

Following these recommendations would justify raising the gravel factor from its current value of 1.4 to a value as high as 2.

Construction Practices

Both the fatigue analyses and the fatigue performance of the asphalt concrete in the HVS tests emphasize the importance of proper compaction of this layer in the pavement structure. Accordingly, compaction requirements should be established to insure that the mix air-voids content at the time of construction should never exceed 8 percent.

While the use of relative compaction requirements based on the laboratory density is satisfactory, a reduction in asphalt content from that selected in the laboratory could lead to a air-void content higher than 8 percent even though the relative compaction requirements are met. Accordingly, the change from a relative compaction requirement to a maximum air void requirement based in ASTM D2041 (“Rice” specific gravity) is strongly recommended.

The pay factor study summarized herein has stressed the importance of proper compaction to insure improved fatigue performance. In addition, the study has highlighted the importance of thickness control of the asphalt concrete layer. Implementation of a bonus/penalty system such as that described herein as a part of QC/QA program has the potential to significantly improve asphalt concrete pavement performance. Thus a “shadowing” study using these pay factors should be instituted.

A weak bond was observed between the first two asphalt concrete lifts in the HVS tests sections. In all cases it was found to significantly degrade pavement performance. While the extent to which weak bonding may be prevalent in California pavements is unknown, the fact that the HVS test pavements were constructed according to standard Caltrans procedures suggests that a weak bond may contribute to performance problems on in-service pavements. If additional investigations confirm such problems, requirements such as the application of a tack coat will result in significant improvements in pavement performance and, hence, reduction in life-cycle cost.

Summary

The information presented herein furnishes examples of the importance of accelerated pavement testing in providing a rapid validation mechanism for new technology. In the three-year period since its inception, the CAL/APT program has focused on a number of problems of importance to Caltrans. As seen in the previous section, a series of recommendations have been developed from the program which involved analytical studies, a laboratory test program, and HVS loading of full-scale (real) pavements constructed at the RFS of UCB.

The program to examine fatigue response of Caltrans designed pavements for a range of California environments has shown that the thickness of the asphalt concrete for heavily trafficked pavements and constructed on weak subgrades may be inadequate, resulting in premature pavement cracking. Use of the mechanistic-empirical analysis and design system described herein should reduce the propensity for such cracking since the results of the HVS tests provide validation for the approach recommended. Moreover, this approach provides the opportunity to explore designs with new binders (e.g. modified materials) and concepts such as the “rich bottom” layer. The system provides a sound basis for exploring long life pavement rehabilitation strategies (LLPRS). Construction variables can be considered within the analysis/design framework providing a logical basis for including reliability as a part of the design process. The efficacy of this approach has already been demonstrated by the development of pay factors for asphalt concrete which could be in the QC/QA program currently underway in California.

The HVS pavement test sections have been constructed according to Caltrans specifications. The results of the tests have conclusively demonstrated the importance of improved asphalt concrete compaction at the time of construction. Longer pavement lives (in terms of ESALs carried) obtained in these pavements point to the efficacy of implementing more stringent compaction requirements as quickly as practicable. If this were accomplished, conservative estimate in terms of cost savings for the Caltrans asphalt concrete overlay program would be of the order of 10 percent of the $\$250 \times 10^6$ expended annually, that is about $\$25 \times 10^6$. Thus, for the same amount of money ($\$250 \times 10^6$), more mileage could be paved and at the same time, the time interval between resurfacing for specific pavement sections could begin to increase.

While the results of the HVS tests on the section with ATPB resulted in longer lives than those for comparably designed pavements containing untreated aggregate, both field experience and analysis suggest that Caltrans should reevaluate the requirements for the use of this material directly under the asphalt concrete. If the decision is made to continue this practice, the results of the laboratory test program examining the behavior of the ATPB in repeated loading while saturated and supported by field performance observations points to the importance of improving the quality of the material through improved mix design practice.

Finally, the project to date has demonstrated the efficacy of government, industry, and academia working together to provide solutions to critical pavement problems. Moreover, it demonstrates the benefits of international cooperation in which technology is transferred among the participants to the benefit of the involved organizations.

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