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CALTRANS ACCELERATED PAVEMENT TESTING (CAL/APT) PROGRAM–TEST RESULTS: 1993-1996

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Abstract. Results to date (Fall 1996) from the Caltrans Accelerated Pavement Testing (CAL/APT) Program are discussed and include those for the Phase I program and initial results from the Phase II program.

The Phase I program involved a study of the comparative performance under Heavy-Vehicle-Simulator-(HVS)–loading of Caltrans dense-graded asphalt concrete (DGAC) and asphalt-rubber, hot mix, gap-graded (ARHM-GG) as overlays on a fatigue-cracked asphalt pavement and an evaluation of pavement rutting from channelized traffic resulting from simulated Automated Vehicle Control Systems (AVCS).

The initial part of the Phase II program includes an evaluation, under HVS-loading, of the performance of two pavements, one containing an asphalt-treated permeable base (ATPB) and the other an aggregate base (AB), designed and constructed according to Caltrans procedures. The HVS testing of pavements for the Phase I program was performed in South Africa while the Phase II program is being conducted at the University of California at Berkeley (UCB) Richmond Field Station.

Laboratory tests have been performed for both phases at UCB involving evaluations of the stiffness and permanent deformation characteristics of the various pavement components and fatigue characteristics of the asphalt mixes. These results have been used to analyze the performance of the HVS-loaded pavements.

By combining the results of both the HVS and laboratory tests, interpretations of pavement performance are described and recommendations for pavement design and construction have been made to Caltrans.

Keywords: Accelerated pavement testing, heavy vehicle simulator (HVS); asphalt concrete; asphalt-rubber, hot mix, gap-graded; asphalt-treated permeable base; fatigue; permanent deformation, channelized traffic.

INTRODUCTION

The Caltrans Accelerated Pavement Testing (CAL/APT) Program was established in 1993 (Nokes et al., 1996). This program, a five-year research and development activity totaling more than $9 million, is a joint effort between Caltrans, the University of California at Berkeley (UCB), the Division of Roads and Transport Technology of the Council of Scientific and Industrial Research (CSIR), South Africa, and Dynatest Consulting Inc. of Ojai, California.

Key to the program are two Heavy Vehicle Simulators (HVS) developed in South Africa. The decision to use the HVS equipment was supported by productive HVS operations by the CSIR, by substantial improvements in South African pavement technology resulting from the HVS program, and by demonstrated expertise of CSIR engineers in full-scale pavement testing based on approximately 25 years of experience and an extensive data base of pavement performance from about 400 HVS test sections.

The program recognizes the importance of complementing full-scale, accelerated-load testing with laboratory testing (Nokes et al., 1996); in particular, results from Strategic Highway Research Program (SHRP) Project A-003A which became available in 1993. These results included equipment and procedures for evaluating the fatigue and permanent deformation characteristics of asphalt-aggregate mixes including the effects of aging and moisture (Monismith et al., 1994). By combining these laboratory innovations with full-scale accelerated pavement testing it is thus possible to develop and implement improved pavement technology. A simplified framework within which this can be accomplished is shown in Figure 1. HVS testing within this framework for the CAL/APT Program includes the use of two HVS units to test pavements, one to test under controlled conditions at the Richmond Field Station (RFS) and the other to test in-service pavements.

To date the program has included (1) A pilot study (termed Phase I), combining HVS testing and laboratory testing within the framework of Figure 1 and completed in early 1994 (Harvey et al., 1994a; Harvey et al., 1994b; Rust et al., 1993; and Weissman, 1993); (2) delivery from the CSIR and acceptance by Caltrans of two HVS units; and (3) completion of four pavement tests on two different pavement structures constructed at the RFS of UCB.

The latter two activities listed above are part of the current 5-year program and are termed Phase II. This paper briefly describes the results of the Phase I study and initial results from the Phase II program.

HEAVY VEHICLE SIMULATOR

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

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 profilometers, road surface deflectometer; crack activity meter; thermocouples, photographic surface crack detector equipment, and a nuclear density gauge.

PHASE I STUDY

The Phase I Project was performed in 1993 by Caltrans, UCB, Dynatest, and the CSIR. Objectives of the project were (1) to demonstrate the capabilities of HVS technology to provide rapid information on pavement performance; (2) to validate the Caltrans asphalt-rubber, hot mix, gap-graded (ARHM-GG) thickness design specification (allowing up to 50 percent thinner layers than conventional dense-graded asphalt concrete, DGAC or AC); and (3) to evaluate pavement rutting from channelized traffic under laterally guided Automated Vehicle Control Systems (AVCS).

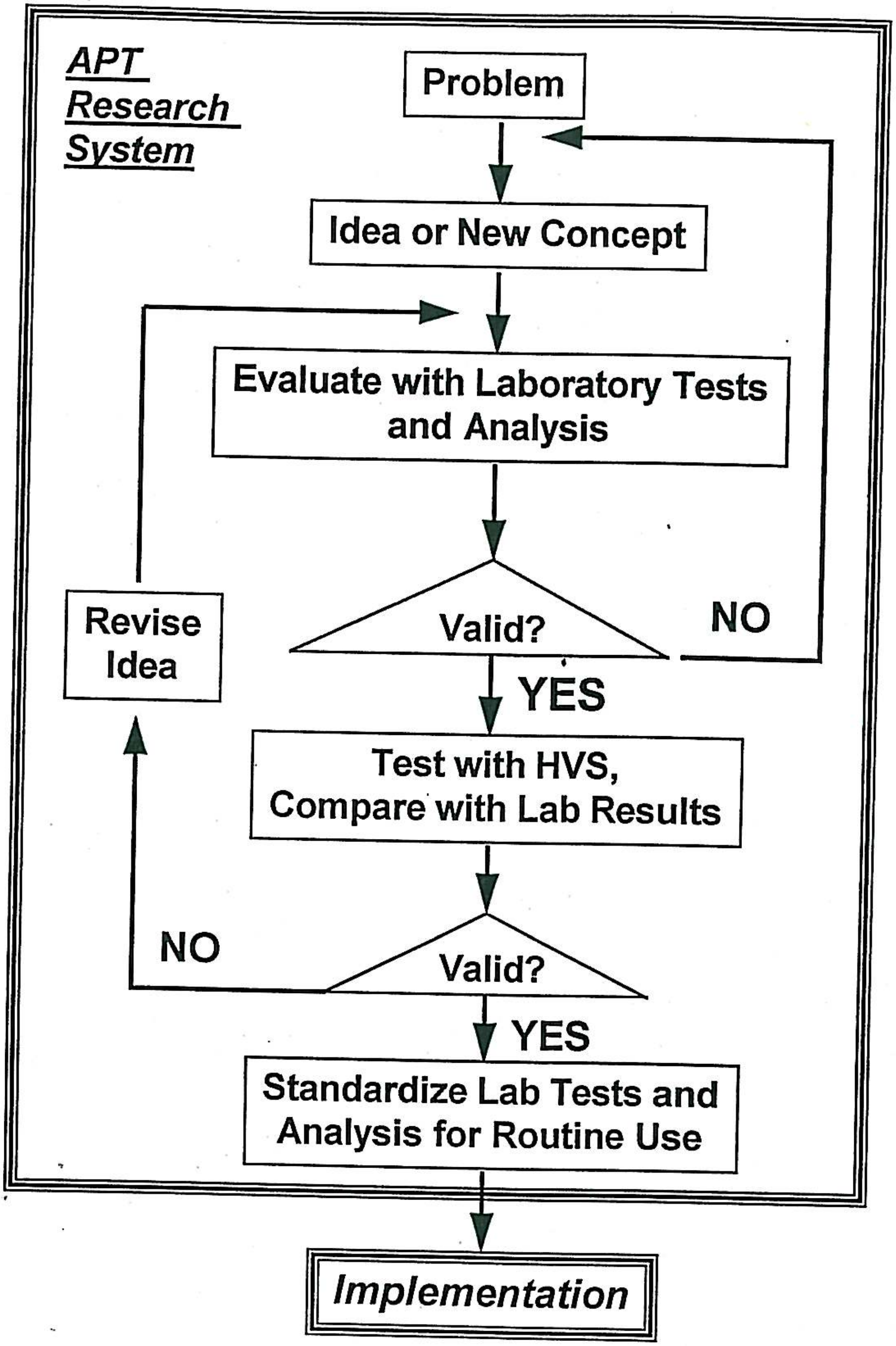

Figure 1. Role of the HVS and Laboratory Testing in Pavement Technology Development and Implementation

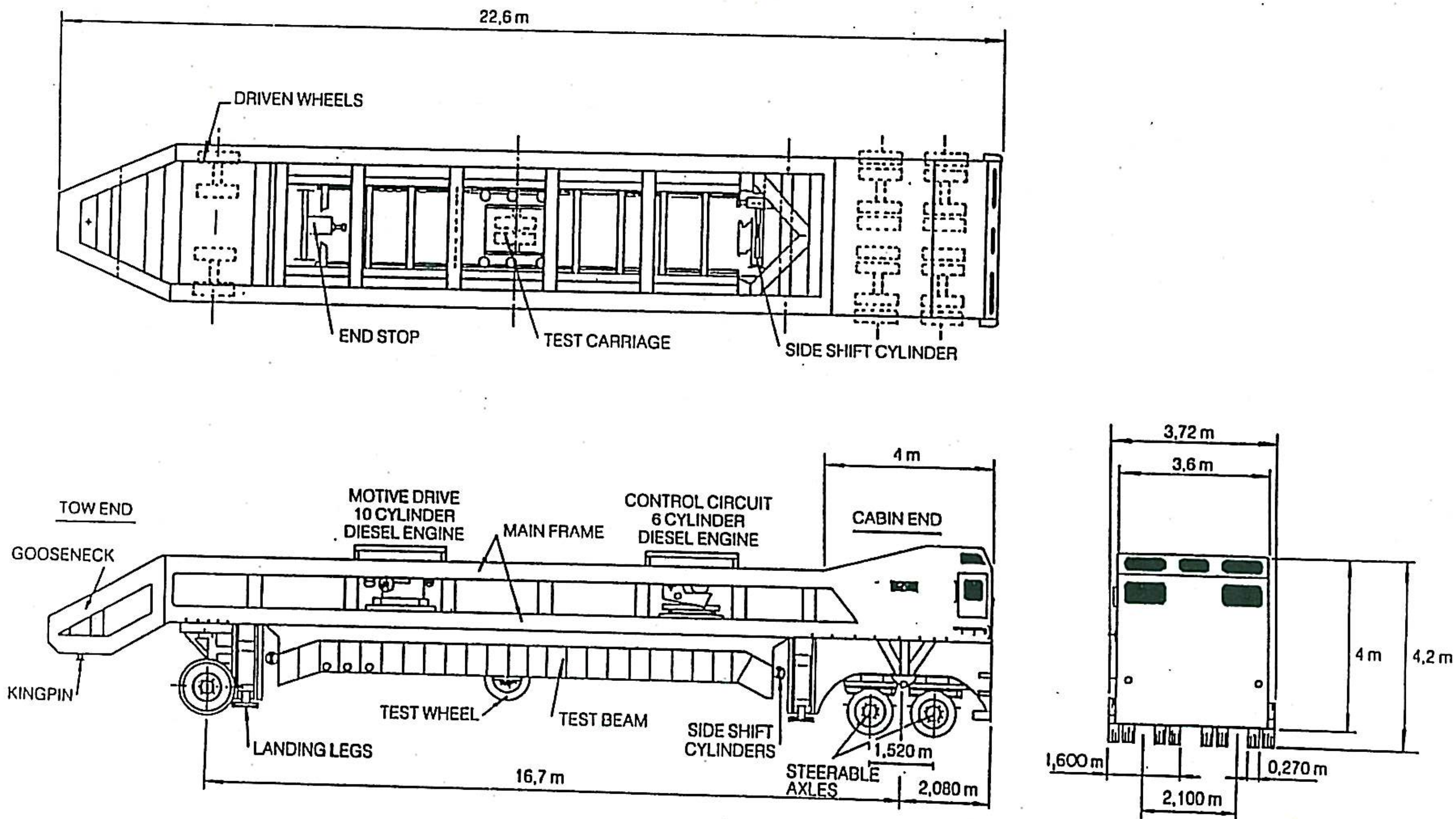

Figure 2. Line Diagram of the Heavy Vehicle Simulator (HVS)

The logistics of the project were complex because test pavements were built in South Africa by a local contractor. CSIR engineers ensured that Caltrans design and construction specifications were met, sampled materials, and conducted HVS tests. Materials samples were shipped to California for testing by UC Berkeley (at their Richmond laboratory) and Caltrans (at the Caltrans laboratory). Dynatest engineers provided substantial coordination and technical support.

The first objective of Phase I was achieved by applying a total of 1,125,000 HVS repetitions (equal to approximately 13.4 million 80 kN [18 kip] Equivalent Single Axle Loads, ESALs) during 66 days of HVS operation, including HVS set-up and movement between sections. HVS results are summarized in Table 1 (showing ESAL equivalency based on a fourth power damage assumption) and described in Rust et al. (1993). To verify HVS results and to provide a link to SHRP A-003A mix test technology, complementary laboratory testing of the asphalt-aggregate mixes included 75 fatigue beam tests to determine fatigue properties, and 23 repetitive simple shear tests and 6 materials characterization tests to determine permanent deformation response (Monismith et al., 1994). The shear test data were used to evaluate rutting predictions from a finite element model developed by Symplectic Engineering, Inc. (Weissman, 1993).

Evaluation of the Comparative Performance of DGAC and ARHM-GG in Overlays. The second objective resulted in validation of the Caltrans thickness design procedure for ARHM-GG overlays. The Caltrans overlay thickness design (1979) procedure evolved from resistance to fatigue cracking observed at field test sections monitored by Caltrans for several years. The design procedure allows ARHM-GG overlays as much as 50 percent thinner than conventional DGAC for the same project conditions and traffic.

A 75-mm (3-in.) DGAC overlay and a 38-mm (1.5-in.) ARHM-GG overlay were built and tested using an HVS while maintaining a surface temperature of 10°C (50°F). ARHM-GG overlays of 25 and 50 mm (1 and 2 in.) were built with the intent of testing one of them, depending on the results from the 38-mm section. The 75-mm DGAC overlay was designed to withstand 172,000 ESALs before cracking, according to the Caltrans standard design procedure. The low traffic was chosen in order to complete HVS testing within the 6-month schedule allowed in the Pilot Project. Results are summarized in Table 2.

HVS results validated the Caltrans overlay thickness design by showing that ARHM-GG can last at least as long, in terms of load-associated cracking, as the thicker DGAC. Complementary lab testing was done to evaluate fatigue (using the SHRP A-003A beam fatigue test) and permanent deformation (using the SHRP A-003A repetitive simple shear test at constant height, RSST-CH) properties of the mixes.

Comparison of the fatigue-life–strain relations from the laboratory flexural fatigue tests for ARHM-GG mixes showed longer fatigue lives than the DGAC mixes at the same strain levels. Predictions of in-situ fatigue life developed from these relations and from maximum tensile strain based on elastic layer theory correlated strongly with HVS test results (Harvey et al., 1994b). Comparison of resistance to permanent deformation show that the ARHM-GG lab mix was less resistant than the DGAC mix to permanent shear deformation at 19°C and 10-percent air voids (Harvey et al., 1994b). However, when compacted to approximately 5-percent air voids, the ARHM-GG lab mix performance was similar to the DGAC mix. Finite element model results of mix performance after 5,000 HVS wheel load repetitions predicted that the ARHM-GG lab mix (with 5-percent air voids) would show less rutting than the DGAC (Weissman, 1993). These predictions agree with the limited RSST-CH testing of the two laboratory prepared mixes.

Pavement Rutting, Channelized Traffic versus Conventional Traffic Wander. Tests associated with the third objective clearly showed that channelization of traffic, which might occur under Automated Vehicle Control Systems (AVCS), can cause significant increases in surface rutting. HVS testing was done on two 100-mm (3.9-in.) DGAC overlays at 25°C (77°F) and 40°C (104°F)—one

Table 1. Summary of HVS Testing Performed by the CSIR for the Pilot Project

Test Section	HVS Wheel Load Magnitude, Repetitions and ESALs		
	Load (kN)	Repetitions Total—1,125,000[a]	Approximate ESALs Total—13,362,844[b]
AVCS —Wandering	70 100	116,000 159,000	1,087,953 6,210,938
AVCS—Channelized	70 100	116,000 48,000	1,087,953 1,875,000
DGAC—75 mm	40 80	175,000 25,000	175,000 400,000
ARHM-GG—38 mm	40 80	175,000 74,000[c]	175,000 1,184,000
ARHM-GG—25 mm	40 80	175,000 62,000	175,000 992,000

Notes:
[a]Number of repetitions equals 1,564,000 if two ends of section (one heated, one not heated) of the channelized and wandering sections are considered separate sections; assumes fourth power.
[b]Number of repetitions is 23,624,688 for the same conditions as note a above.
[c]Approximately 12,000 repetitions at -5°C.

Table 2. Cracking on Conventional and ARHM-GG Overlays in the Pilot Project

Load Repetitions	Wheel Load	Overlay Test Sections		
		AC Overlay (75 mm)	ARHM-GG (38 mm)	ARHM-GG (25 mm)
0 to 100,000	40 kN	Fine cracks at 100,000	No cracks visible	No cracks visible
100,000 to 175,000	40 kN	Block cracks at 175,000	No cracks visible	No cracks visible
		Wheel load changed to 80 kN		
175,000 to 200,000	80 kN	Completely cracked	No cracks visible	Fine cracks
200,000 to 237,000	80 kN	Stopped test	No cracks visible	Completely cracked
		Surface temperature dropped to -5°C		
237,000 tó 250,000	80 kN	Stopped test	One-half of section cracked	Stopped test

tested with an approximately normal lateral distribution of traffic across 1 m (3.3 ft) and the other tested with the HVS wheels locked in one lateral position. HVS dual-wheel loads of 70 kN (15.7 kip) were applied for 116,000 repetitions at both temperatures and following both lateral distributions. At the end of this initial loading, on the 25°C test section, rut depths were 4 mm in the "channelized" section and 3 mm in the "wandering." At the 40°C test section, the rut depth was 13 mm in the "channelized" section versus 7 mm in the "wandering" section. The dual-wheel load was increased to 100 kN (22.4 kip) and then 48,000 repetitions were applied to the "channelized" sections, which resulted in 2 mm of additional rutting at both temperatures. In the "wandering" sections, 159,000 repetitions were applied, which caused additional rutting of 4 mm and 6 mm in the 25°C and 40°C sections respectively. Complementary lab testing found that decreasing the air voids of the DGAC resulted in nearly five times more load repetitions being required to reach a given permanent shear strain.

Table 3. Elastic Constants for Plastic Branch

Constant	ARHM-GG	DGAC
C_1	1.0500E+4	7.2500E+3
C_2	-4.3013E+2	-1.8785E+2
C_3	1.0000E+5	-1.0000E+5
C_4	4.2500E+5	5.7500E+4
C_5	·1.8250E+5	1.2500E+5
C_6	1.8750E+7	2.2500E+6
C_7	-7.7500E+6	-9.7500E+6
C_8	4.5000E+7	1.8500E+7
C_9	1.5000E+7	6.5000E+6

Laboratory tests were performed to define the characteristics of the DGAC according to the constitutive relationship shown schematically in Figure 3 (Weissman, 1993). These tests had been developed as a part of the SHRP A-003A project using the simple shear test device (Sousa et al., 1994). The resulting properties are shown in Tables 3, 4, and 5. Tables 3 and 4 contain the parameters used to define the non-linear elastic and plastic components of the constitutive relationship depicted in Figure 3 while Table 5 contains the parameters used to define its viscoelastic characteristics.

Table 4. Plastic Parameters

	ARHM-GG	DGAC
σ	9.0000E-1	2.5000E-1
β	5.0000E-1	5.0000E-1
$\overline{H}$	1.7500E+3	7.5000E+2

Plane strain simulations of the pavement section shown in Figure 4[1] were conducted. The pavement was subjected to 5,000 cycles of loading (0.15 sec. haversine pulse of 530 kPa [76.9 psi] applied over a width of 293 mm [11.5 in.], followed by a rest period of 4.7 sec). The layer thicknesses and properties are summarized in Table 6. While the subgrade thickness is shown in Table 6 as infinite in extent, it was assumed to be 600 mm (23.6 in.) thick for the finite element idealization since this depth was considered sufficient for analysis.

The increase in rut depth with load cycles is shown in Figure 5 while the deformed shape of the finite element idealization of the pavement is shown in Figure 6 after 5,000-load applications. For comparison, the actual rutting observed is shown in Figure 7. The actual and computed rut depths are different because time limitations precluded carrying out the simulation to the actual number of load applications. However, the shapes of the predicted and actual deformed sections are strikingly similar. Thus the field measurements lend credence to the analytical procedure adopted and suggests further developments using this approach should improve the evaluation of permanent deformation characteristics of mixes.

[1] Numbers associated with each of the layers are shown in Table 6.

Table 5. Maxwell Elements' Properties

Element #	Characteristic time (λ), sec.	ARHM-GG spring (μ), psi	DGAC spring (μ), psi
1	1.0E+5	3.0505E+2	1.4682W+2
2	1.0E+4	1.2760E+2	8.2697E+1
3	1.0E+3	1.8202E+2	1.3001E+2
4	1.0E+2	3.3927E+2	3.2520E+2
5	1.0E+1	1.685E+3	1.8491E+3
6	1.0E+0	1.4514E+4	5.8188E+3
7	1.0E-1	5.1460E+4	2.5047E+4
8	1.0E-2	2.7446E+5	1.3358E+5

Table 6. Pavement Properties-Stiffnesses and Thicknesses

Layer	**Thicknesses mm (in.)**	**Stiffness MPa (psi)**	**Poisson's Ratio**
asphalt concrete (1)	100 (4.0)	See Tables 3,4,5	
old seal (2)	40 (1.6)	71.3 (10,350)	0.30
natural sandstone gravel (3)	215 (8.5)	306 (44,350)	0.35
weathered ferri crete (4)	90 (3.5)	204 (29,600)	0.35
weathered shale (5)	280 (11.0)	112 (16,300)	0.35
subgrade (6)	inf.	71.3 (10,350)	0.35

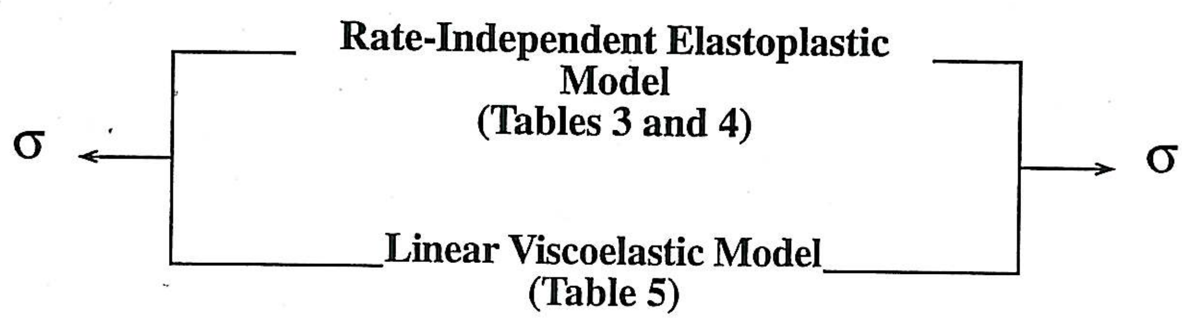

Figure 3. Constitutive Relationship Concept, One Dimensional Schematic

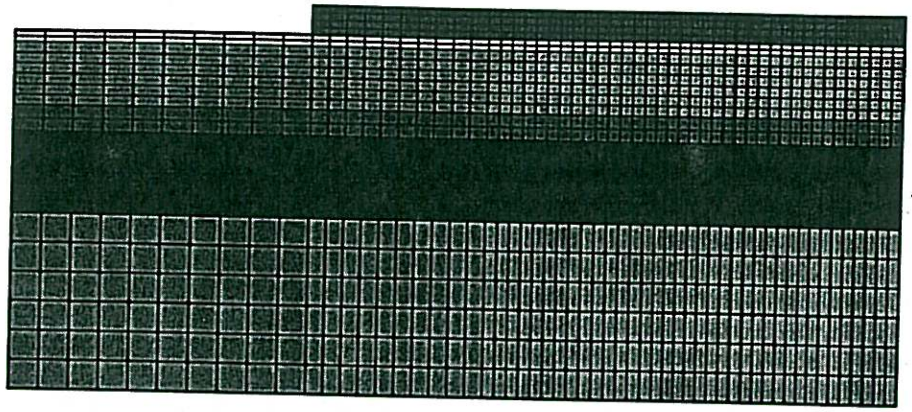

Figure 4. Finite Element Idealization—Channelized Traffic HVS Test Section

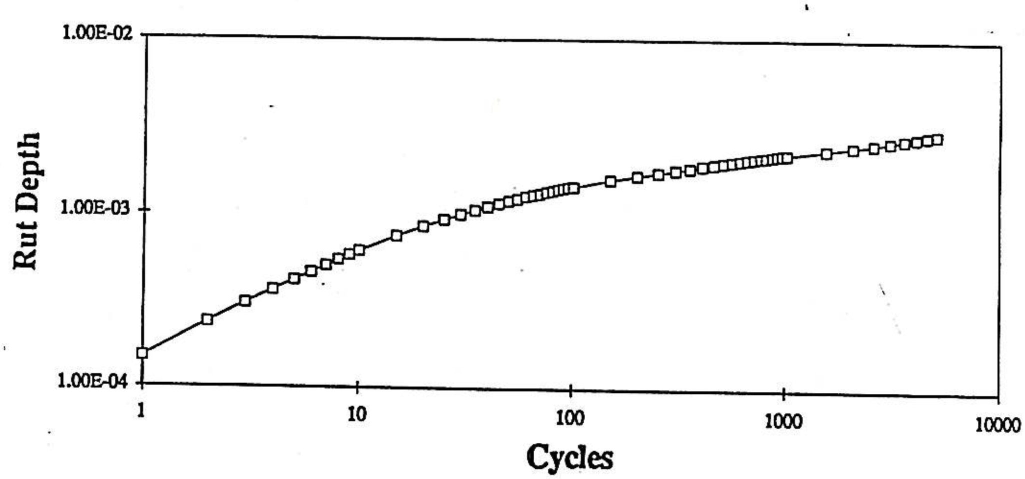

Figure 5. Rut Depth (in.) vs. Number of Load Applications from Finite Element Analysis

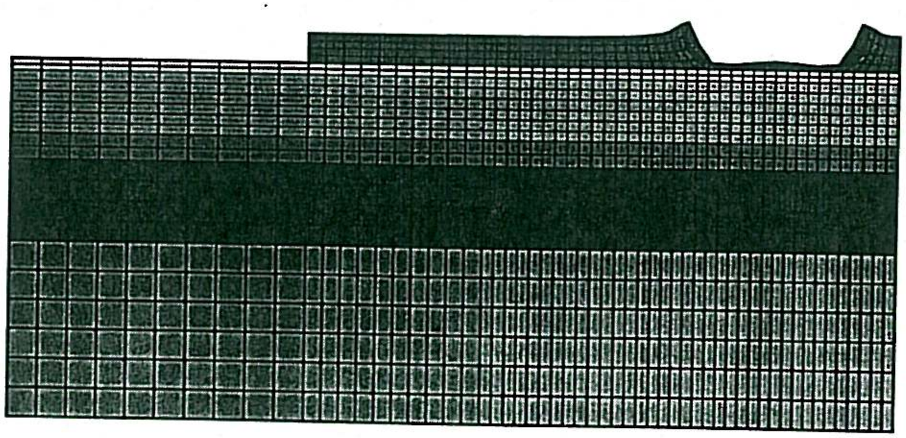

Figure 6. Deformed Mesh (amp. 2,000 times vertically) DGAC Mix-Channelized Traffic

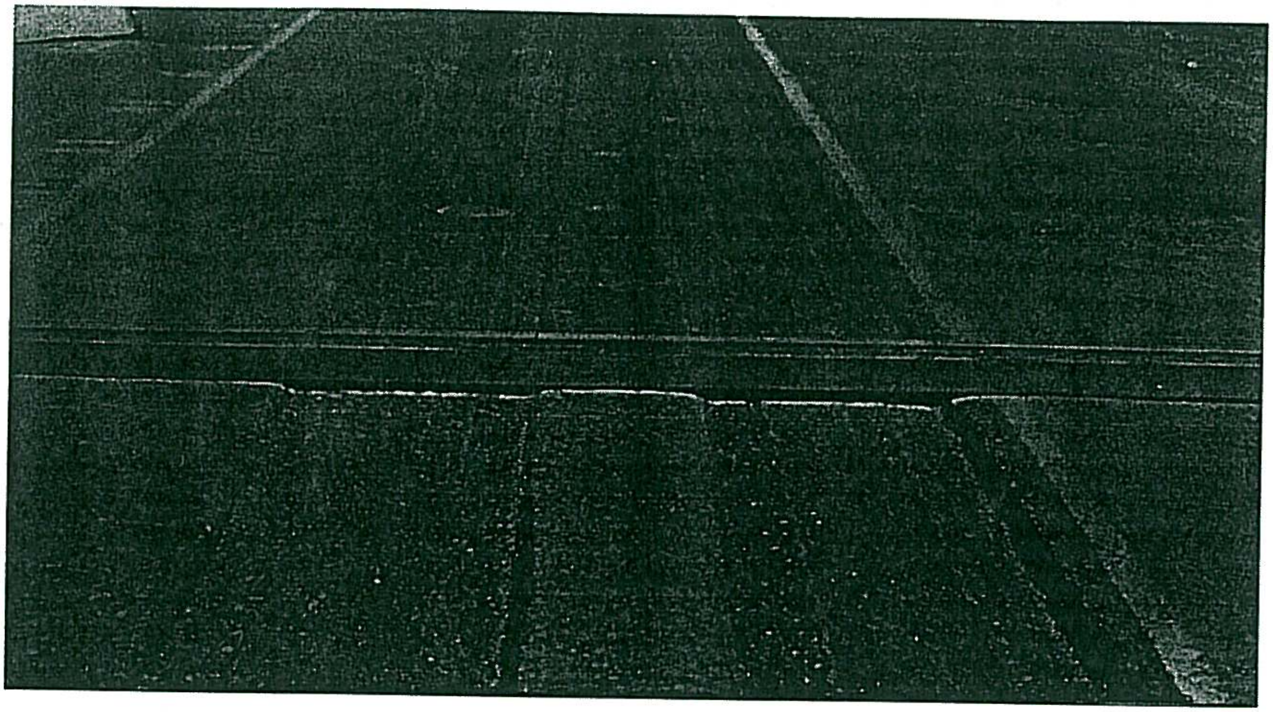

Figure 7. Rutting after Channelized Traffic

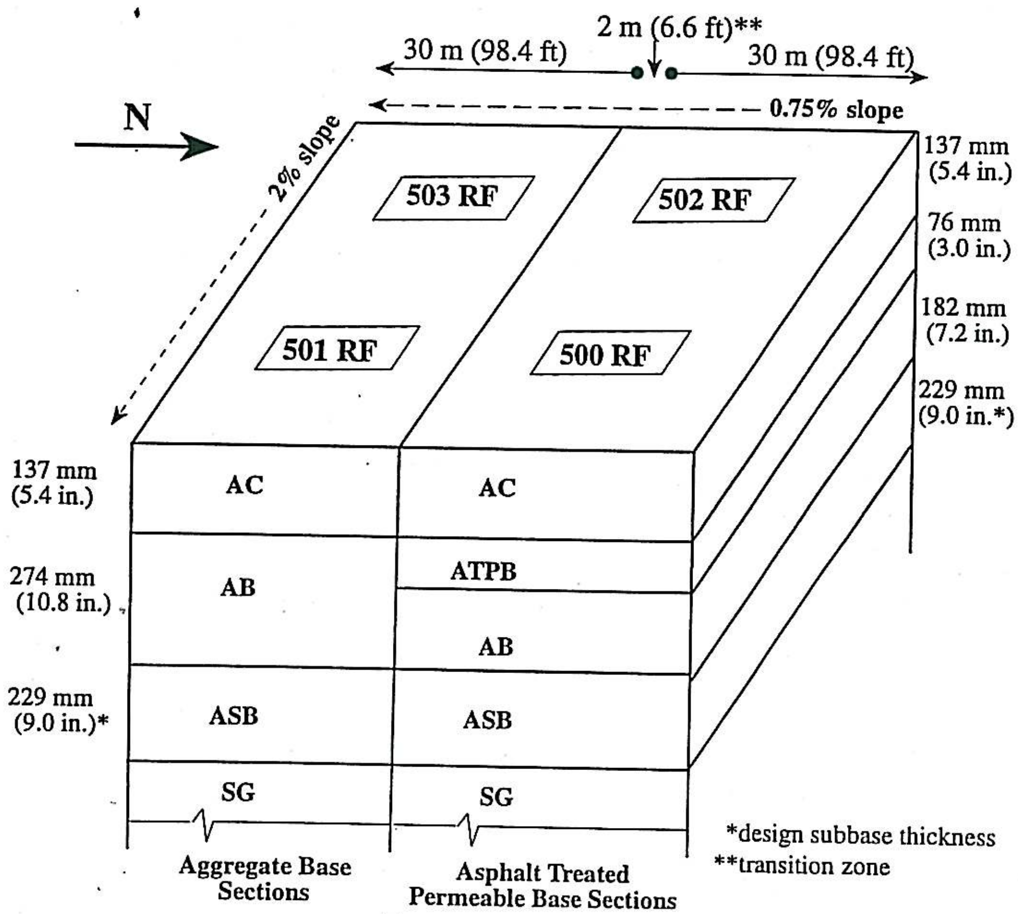

Figure 8. Test Pavements

PHASE II STUDY—INITIAL RESULTS

The first Test Plan for the Phase II activities includes 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 (2) to compare the fatigue performance of structural overlays of asphalt-rubber, hot mix, gap-graded (ARHM-GG) mixes using type 2 asphalt-rubber binder with that of conventional dense-graded asphalt concrete (DGAC) mixes. This section describes the results of two HVS tests–one on a section containing the ATPB and the other on a section containing the AB. Both sections were tested without adding water because of unpredictable water movement. It is expected that the sections will be tested under wet conditions in the future.

Harvey et al. (1996a) contains detailed information on the construction details as well as results of tests to define the properties of the components of the pavement sections. Results of stiffness tests on the ATPB are included in (Harvey and Tsai, 1997) while stiffness tests on the AB and aggregate subbase (ASB) are included in Harvey et al. (1996a).

Pavement Sections. The pavement structures are shown in Figure 8. They consist of a clay subgrade (AASHTO A-7-6), ASB, AB, ATPB, and asphalt concrete (AC). Thicknesses for the pavement sections were selected according to the Caltrans design procedure (1990) for a Traffic Index of 9 (800,000 to 1,200,000 ESALs) and a design "R" value for the subgrade of 10 (measured range of 4 to 30) (Harvey et al., 1996a).

The test pavements were constructed according to Caltrans specifications (1992). When the construction of the test pavements had been 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. A weakness in (and even a lack of) the bond between the two asphalt-concrete lifts was observed. The influence of this poor bond will be considered subsequently in this section.

Traffic Loading with HVS. HVS trafficking of the ATPB section commenced on May 3, 1995, and was completed on November 8, 1995. A total of 2.57×10^6 load repetitions were applied during this period 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. Trafficking of the AB section commenced on November 20, 1995, and was completed on February 26, 1996. The same loading sequence was followed for the AB section with a total of 1.43×10^6 load repetitions applied.

In the ATPB section the first load-associated (fatigue) cracks were observed at approximately 7×10^5 load repetitions. At 2.57×10^6 repetitions, alligator-type cracking had reached a level which, according to Caltrans pavement management criteria (1993), constituted fatigue failure. In the AB section, cracks first appeared at about 6.5×10^5 repetitions and loading was terminated at 1.43×10^6 load repetitions when the amount of alligator-type cracking visible on the surface had surpassed that on the ATPB section.

Material Characteristics and Pavement Modeling. To estimate the propensity for fatigue cracking in the pavement test sections the structures were represented as layered elastic systems and the computer programs ELSYM5 (Ahlborn, 1972) and CIRCLY (Wardle, 1976) were utilized.

The program CIRCLY was used to address the weakness observed in the bond between the two layers of asphalt concrete. The ELSYM program (as currently formatted) permits only full friction at the interfaces in the asphalt-concrete layers; on the other hand, CIRCLY has provisions for both full-friction and frictionless interfaces.

The lack of bond was confirmed through a number of cores and slabs taken from both test sections at the conclusion of the loading tests to check changes in the densities of the lifts. When these cores were removed it was noted the two lifts were unbonded and that there was evidence of movement between the lifts resulting from the

Table 7. Material Characteristics of Pavement Layers

DRAINED PAVEMENT					
Pavement Layer	Thickness (mm)	Elastic Modulus (psi)			Poisson's Ratio
		Minimum	Middle Values	Maximum	
Asphalt concrete top lift	61		8268		0.35
Asphalt concrete bottom lift	76		10335		0.35
Asphalt treated permeable base (ATPB)	76	689		1034	0.4
Aggregate base and subbase	310 401 488	103	138 189 242 293	448	0.35
Subgrade	Semi-infinite	17	69	234	0.45

UNDRAINED PAVEMENT					
Pavement Layer	Thickness (mm)	Elastic Modulus (MPa)			Poisson's ratio
		Minimum	Middle values	Maximum	
Asphalt concrete top lift	61		8268		0.35
Asphalt concrete bottom lift	76		10335		0.35
Aggregate base	274	172	365	448	0.35
Aggregate subbase[a]	127 218 305	103	242	345	0.35
Subgrade	Semi-infinite	17	69	234	0.45

[a]The thickness of the aggregate subbase at the test sections is as follows: 5.0 in. (127 mm) and 8.6 in. (218 mm) in the ATPB sections and 8.6 in. (218 mm) and 12.0 in. (305 mm) in the AB sections. The sections whose data are reported herein have subbases of 5.0 in. (ATPB) and 8.6 in. (AB).

deflection of the pavements under load. Also, it was observed that the cracking observed at the surface existed only in the top lift; in each section the lower lifts appeared to be intact.

Moduli for all the pavement layers were obtained from a number of different measurements both through in-situ testing of the pavement sections and from laboratory tests on representative specimens of the various materials and are summarized in Harvey et al. (1996a).

For the unbound layers, triaxial compression repeated-load tests, bender element analysis, k-mold tests, dynamic cone penetrometer tests, and backcalculations from heavy weight deflectometer (HWD) deflections were used. Triaxial compression repeated-load tests were used to estimate the modulus of the ATPB. Initially the moduli of the ATPB were estimated and are shown in Table 7. Measurements from repeated-load tests indicated the moduli to be in the range 800 to 2,000 MPa which are higher than the values shown in the table. Moduli for the asphalt concrete were determined from flexural-beam tests (Harvey et al., 1996a) and by backcalculation from the HWD measurements.

Table 7 contains a listing of the stiffness values considered to be representative of the range in properties for each of the layers. It will be observed that stiffnesses of the asphalt-concrete lifts differ somewhat resulting from the fact that the lower lift exhibited a higher degree of compaction (as measured by smaller air-void content).

While the range in moduli shown in Table 7 were used to make the preliminary estimates of performance reported in Harvey et al. (1996), the analyses reported in the next two sections are based on moduli determined from stiffness measurements during HVS-loading. To accommodate differences in subbase thicknesses due to the designed crossfall, different subbase thicknesses were modeled (Table 7). As will be seen subsequently these values fall within the ranges shown in Table 7.

Comparison of Actual and Predicted Performance, ATPB Section. Trafficking on the ATPB test section was stopped at 2.57×10^6 repetitions when extensive surface cracking met Caltrans failure criteria. Predictions of the fatigue performance of this test section, summarized in Harvey et al. (1996), covered a wide range of fatigue lives using four different sets of fatigue criteria. Of these four, only the Asphalt Institute fatigue model and the fatigue model developed using the SHRP laboratory fatigue beam test resulted in predictions that approximate the observed fatigue life. Hence they were used for the analysis presented herein.

The moduli shown in Table 8 were used to model the pavement behavior:

Table 8. Pavement Structure, Characterization

Layer	Modulus—MPa (psi)
Asphalt concrete top layer	8268 (1,200,000)
Asphalt concrete bottom layer	10,335 (1,500,000)
Asphalt-treated permeable base	1034 (150,000)
Combined aggregate base and subbase	189 (27,500)
Subgrade	69 (10,000)

With these moduli and assuming a full-friction bond between the two AC lifts and the fatigue relationship developed from the SHRP laboratory flexural fatigue test, the predicted fatigue life is 5.08×10^6 repetitions. The loading sequence for this estimated life is based on 150,000 repetitions of a 40-kN load, 50,000 repetitions of a 80-kN load and the remaining repetitions with the 100-kN load. Cumulative damage for the three loads is based on the linear-sum-of-cycle-ratios hypothesis. The fatigue life predicted by the Asphalt Institute model is fractionally larger, i.e. 6.07×10^6 repetitions.

From the analyses reported in Harvey et al. (1996a) the ratio of the fatigue life assuming a full-friction interface to that with a frictionless interface is approximately 2.5. Accordingly, the fatigue life for the frictionless case and the moduli listed above is estimated at 2.03×10^6 repetitions. Since the interface between the lifts is neither represented by a full-friction bond nor a frictionless bond, it is expected that the actual fatigue life would be somewhere in between the predictions for each of these cases. The observed fatigue life of 2.57×10^6 repetitions lies within the 2.03×10^6 to 5.08×10^6 range. Accordingly, it is concluded that fatigue response measured in the laboratory test program and adjusted by the shift factor of 13 (as suggested by the SHRP research [Tayebali et al., 1994]) provides a reasonable prediction of fatigue life. For the particular mix used, the Asphalt Institute design equation also provides reasonable fatigue predictions.

Comparisons have been made of the ESALs actually carried by the pavement to those for which it was designed. According to Caltrans conversion,

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

the applied 2.57×10^6 load repetitions are equivalent to 112×10^6 ESALs. This corresponds to a Traffic Index of 15.8. In Harvey et al. (1996a), based on considerations of fatigue in the asphalt concrete, it was theoretically determined that an exponent of 2.5 rather than 4.2 appeared to reasonably express the relative damage caused by wheel loads of different magnitudes for this mix. Using the 2.5 factor, a total of 24.1 million ESALs is calculated. This corresponds to a Traffic Index of 13. Regardless of the conversion, it is evident that the section carried more traffic than that for which it had been designed.

This difference could be due to at least three factors. First, the fatigue response of the mix was better than would be obtained in actual pavements constructed according to current Caltrans specifications which are based on a percentage of laboratory-compacted mix density and typically permit air-void contents of 8.5 percent or more. The air-void content of the top lift after construction was about 7.5 percent and about 4.0 percent in the bottom lift. Second, the ATPB remained dry throughout the loading program and therefore contributed to the improved performance because of its higher value of stiffness as compared to the untreated aggregate base. Current Caltrans design methodology assumes no structural advantage when AB is replaced with ATPB. Third, the test was completed during a period of no rain. Limited test data indicate that the AB appeared to increase in stiffness, likely due to drying. This factor could also provide some contribution to reduced strains in the AC and, therefore, increased repetitions before cracking.

The fatigue life for the pavement would have even been longer than observed had there not have been a reduction in the bond between the two lifts of AC. Analyses indicate that the reduced bonding between lifts and the desirably low air-void content in the lower lift resulted in the critical tensile strain occurring on the underside of the top lift. This suggests that the observed cracking started on the underside of the unbonded top lift and propagated to the surface. The cores from the trafficked section confirm that cracking appears to have occurred only in the top lift.

Rutting was also observed in the ATPB test section. The average maximum rut depth at the centerline at the end of the test was 15 mm. From the multi-depth deflectometer measurements it was determined that about 52 percent of the rutting occurred in the AC layer.

Measurements of densities on cores from the trafficked portion of the pavement indicated that traffic compaction was limited to the upper lift of the asphalt concrete. In this lift the air-void content was reduced by about 2 percent (from about 7.5 percent to 5.5 percent). Of the deformation occurring in this layer, at the most about 1.5 mm can be attributed to volume decrease (i.e. the 2-percent reduction in air-void content noted). Thus the additional permanent deformation is likely due to shear deformations occurring in this upper lift as suggested by the surface profiles shown in Figure 9 at point 15. Results of the simple shear tests on the AC reported in Harvey et al. (1996a) support this explanation.

The repetitive simple shear tests at constant height (RSST-CH) were performed on cores from the test pavement and on specimens compacted by rolling wheel compaction. Results of the simple shear tests on these specimens indicated less resistance to permanent deformation of the less well compacted (higher air-void content) mix in the top lift. It must be emphasized, however, that the percentage air voids is not the only factor which controls the resistance to permanent deformation in asphalt concrete; gradation and

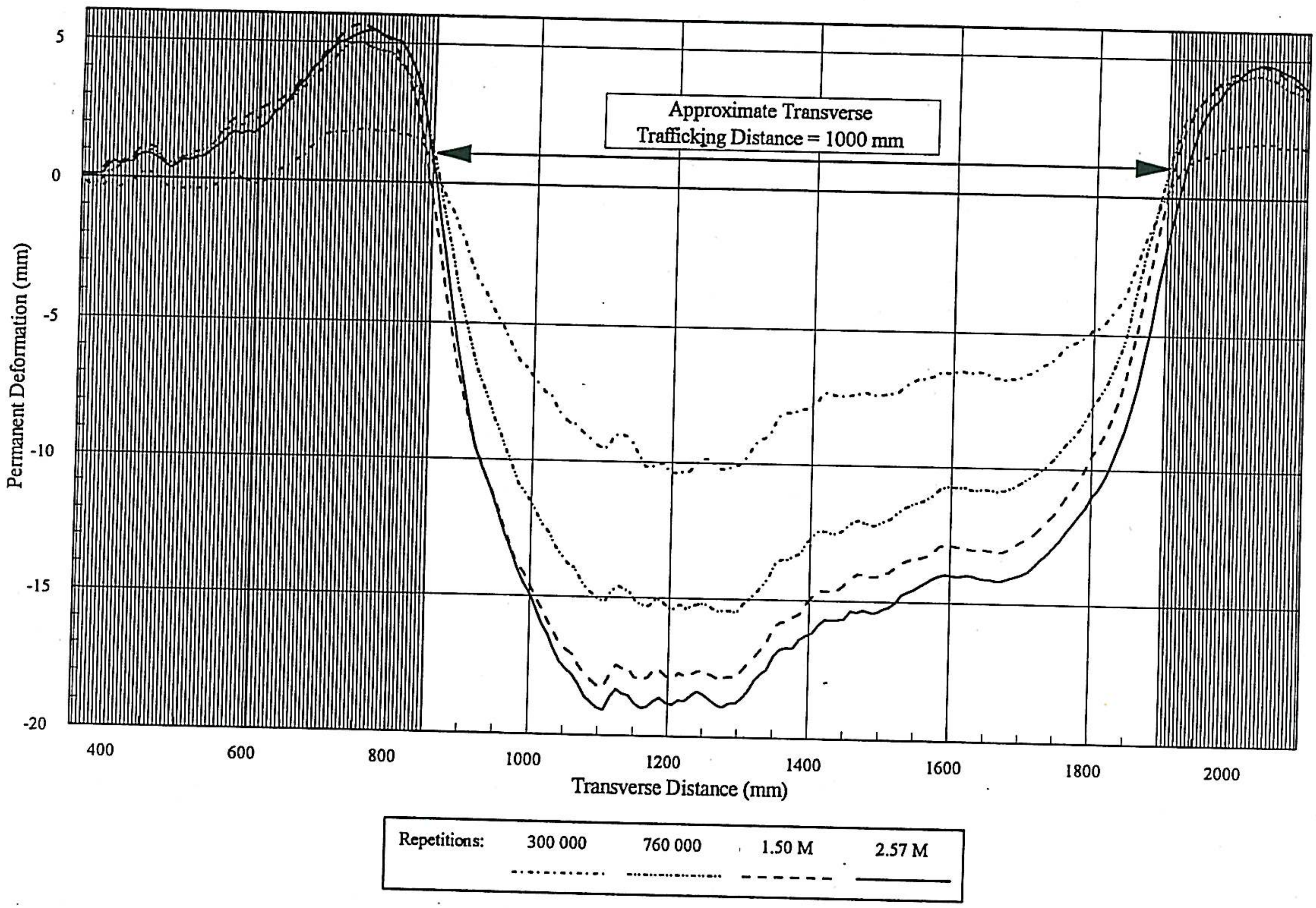

Figure 9. Point 15 Profilometer Cross-section and Final Straight Edge Measurement

quality of aggregate, type and quantity asphalt also play an important role. However, the positive effects of properly compacted mixes are evident. Ensuring good compaction of asphalt concrete during construction led to improved rutting performance.

Comparison of Actual and Predicted Performance, AB Section. In the AB test section trafficking was stopped at 1.43×10^6 repetitions. The observed cracking was somewhat greater in extent than for the ATPB section and was within the limits defined by Caltrans (1993). As with the ATPB section, cracking was observed only in the top lift of the asphalt concrete.

Analyses similar to those reported in the previous section were performed. Because this test was conducted during the rainy season, a different modulus was used for the AB/ASB layer to simulate the pavement response during the 100-kN loading than used for the ATPB section. Material characteristics and thicknesses used for the analyses are summarized in Table 9. The values shown in Table 8 were selected to produce deflections which are representative of those measured with the road surface deflectometer (RSD) under the various loading conditions.

As with the ATPB, the linear-sum-of-cycle ratios was used to sum the effects of the damage produced by three load levels. For the full-friction case the produced fatigue life using the laboratory-determined fatigue response was 2.1×10^6 repetitions while the life assuming a smooth interface was 0.87×10^6 repetitions. These results bracket the number of repetitions, 1.43×10^6, when the section was considered to have failed.

This section, like the ATPB section was designed for a Traffic Index of 9. Applying the Caltrans equivalency factor of 4.2 the number of applied ESALs is estimated to be 58.8×10^6, which corresponds to a TI of 14.6. Alternatively, if the value of 2.5 is used for this conversion then the section sustained 12.6×10^6 ESALs, a TI of 12.2. As with the ATPB section, the actual ESALs exceed the design ESALs, in this case by a factor ranging from 12 to about 50. Had this section maintained its integrity, that is, no slippage at the interface between the 2 AC lifts, it is likely that it would have carried a larger number of repetitions—probably approaching the 2×10^6 repetitions estimated using the ELSYM5 program and the fatigue relationship developed for the SHRP developed laboratory flexural fatigue test. As with the ATPB section, the improved performance is due in part to greater fatigue resistance of the mix brought about by better compaction.

Effect of Temperature Environment on Fatigue Performance of HVS Test Sections at RFS. Of interest in the analysis and interpretation of HVS test results is the influence of the temperature environment on fatigue performance and the impact of the relatively controlled environment of the HVS Test Sections at RFS on pavement life as compared to in-situ environments occurring in California. To assess this temperature influence, a number of simulations were performed using four temperature environments which included the pavement temperatures measured during the ATPB test as well as pavement temperatures (simulated by a software program CMS4 [Harvey et al., 1996b]) for three typical California environments: coastal, mountain and desert. The period of analysis for the ATPB section spanned June 2, 1995 when the 100-kN load trafficking began, to the end of the test on November 8, 1995. For the three California environments the same dates were used in the analysis but the data were obtained from 1988, a randomly chosen year for which the necessary weather data were readily available.

Because limited fatigue test data over a range of temperatures for the actual mix were available at the time of analysis, AC stiffness and fatigue life relationships developed earlier were used (Harvey et al., 1996b). The simulations assumed that the critical strain location was at the bottom of AC lift at the centerline of the dual-tire assembly. ELSYM5 was used to determine the critical tensile strains under dual tires having a combined load of 100 kN with center-to-center spacing of 366 mm and tire pressures of 690 kPa. Characteristics of the pavement structure are shown in Table 10. Full friction was assumed

Table 9. Pavement Structure and Material Characteristics, AB Test section

Pavement Layer	Thickness mm (in.)	Poisson's Ratio	Stiffness MPa (psi)	
			40 kN, 80 kN	100 kN
AC Top Lift	74 (2.9)	0.35	8,268 (1,200,000)	8,268 (1,200,000)
AC Bottom Lift	76 (3.0)	0.35	10,335 (1,500,000)	10,335 (1,500,000)
AB/ASB	503 (19.8)	0.35	189 (27,500)	110 (16,000)
Subgrade		0.45	69 (10,000)	69 (10,000)

Table 10. Pavement Structure and Material Characteristics

Pavement Layer	Thickness (mm)	Poisson's ratio	Stiffness (MPa)
AC Top lift	74	0.35	varies with temperature
AC Bottom lift	76	0.35	varies with temperature
ATPB	76	0.4	1200
Aggregate base/subbase	310	0.35	242
Subgrade		0.45	69

between the top and bottom AC lifts for this analysis.

Simulated fatigue damage was accumulated on an hourly basis. For each hour, the average temperatures in the top and bottom lifts were first tabulated. AC layer stiffnesses, determined from these temperatures and the AC stiffness-temperature relationship, were used to compute the critical tensile strain at the bottom of the bottom lift. Combination of bottom-lift temperature and maximum tensile strain enabled estimating the fatigue life. The relative amount of damage accumulated during each hour was calculated from the product of the reciprocal of fatigue life and the number of load repetitions. Simple summation produced the total damage accumulated during the simulation period. The accumulated fatigue damage totals as a fraction of total fatigue life are as follows:

HVS at RFS	0.183
Coastal	0.154
Mountain	0.131
Desert	0.067

While it is important to recognize that this was a comparative analysis, the totals suggest that the most damaging environment with respect to fatigue distress is the enclosed environment at the RFS, followed by the coastal, mountain and desert environments. Similar investigations, based on a range of material characteristics or loading conditions, were completed and also resulted in the rankings listed above. At the HVS site the majority of the temperatures fall within the critical range for fatigue (15° to 30°C) (Deacon et al., 1994) resulting in more fatigue damage per repetition than the other three sites where less of the hourly temperatures fall within the critical range.

SUMMARY AND CONCLUSIONS

Summary. In the period since July 1994, two HVS units have been delivered and accepted by Caltrans and four test sections on two pavement sections have been completed (as of September 1996). Both HVS's have been utilized to test the pavement sections at the RFS with an estimated total of about 350×10^6 ESALs applied (using the Caltrans factor of 4.2). A comprehensive laboratory program is underway to examine the fatigue and permanent deformation characteristics of binder-aggregate mixes and to evaluate SHRP developments including aspects of the Superpave technology.

The importance of the link between the laboratory test program and HVS testing has been well illustrated by both Phase I and Phase II studies. The results of the laboratory test program have demonstrated the effects of the degree of compaction (as measured by air-void content) and asphalt content on fatigue performance of asphalt mixes (Harvey et al., 1996a). Analysis of the results of the HVS test results for the two pavement sections support the findings of the laboratory test program.

Similarly in the permanent deformation area, the results of the HVS tests (particularly Phase I as reported herein) support the direction of research to develop a constitutive relationship which can be used in a finite element analysis to estimate the development of permanent deformation in the asphalt-aggregate layer. Analyses similar to those reported herein for Phase I are underway for the Phase II study. Results like those presented in Figure 9 suggest, however, that development of the model depicted in Figure 3 should continue.

CONCLUSIONS

The results of the Phase I study have provided validation of the Caltrans overlay thickness by showing that an ARHM-GG mix can last at least as long, in terms of load-associated cracking, as a thicker DGAC. In addition, this study clearly demonstrated that channelization of truck traffic which might occur under AVCS can cause significant increases in surface rutting as compared to conventional highways where some wander of truck traffic takes place.

Results of the Phase II study which have been completed and analyzed at this time indicate that the pavement sections containing the ATPB and AB carried substantially more traffic than the amount for which they were designed. The differences are likely due to a number of factors including:

a) The fatigue response of the mix used in the two pavements was better than expected for in actual pavements constructed according to current Caltrans specifications which are based on a percentage of laboratory-compacted mix density.

b) The ATPB remained dry throughout the loading program and therefore contributed to the improved performance because of its higher value of stiffness as compared to the untreated aggregate base.

c) Limited data suggest that the effect of water on the pavement performance was not as severe as may have been anticipated in the initial design. This factor could also reduce strains in the AC and, therefore, increased repetitions before cracking.

It is likely that the fatigue life for the pavements would have even been longer than observed had there not have been a reduction in the bond between the two lifts of AC. Analyses indicate that the reduced bonding between lifts and the desirably low air-void content in the lower lift of each pavement resulted in the critical tensile strain occurring on the underside of the top lift. This suggests that the observed cracking started on the underside of the top lift and propagated to the surface. Cores showed cracks only in the top lift.

Temperatures during the tests resulted in a more severe condition for fatigue cracking than would be obtained in representative coastal, desert, and mountain environments in California.

While it has not yet been possible to examine the pavement sections by excavating a test pit, it is likely that the rutting which occurred in the AC was limited to the top 75 mm (3 in.) in both sections. Examination of the cores extracted from the ATPB section showed that the majority of the AC rutting took place in the upper lifts in part evidenced by a 2-percent reduction in air-void content which occurred in this lift during trafficking. Moreover, the shear resistance of the mix in the upper lift was lower than that of the same mix in the lower 75 mm (3 in.).

Based on the results of these tests it would appear desirable for Caltrans construction procedures to be modified. The following two recommendations have been made to Caltrans (Harvey et al., 1996c).

a) The Caltrans construction requirements should be changed to insure improved compaction of the asphalt concrete. Currently the degree of compaction is specified in terms of a percentage of the laboratory density obtained with 150 tamps of the kneading compactor. This permits air voids at the time of construction as high as 10 to 11 percent. Based on the evidence obtained from this study as well as elsewhere, it is strongly recommended that compaction control be based on the theoretical maximum specific gravity of the mix (as determined by ASTM D2041) and that the air-void content at the time of construction not exceed a value of 8 percent. By changing the compaction requirement to conform to this recommendation substantial increase in pavement performance is likely.

b) While not as clearly demonstrated as the effects of compaction, it is evident that consideration should be given to improving the bond between lifts during the compaction of asphalt-concrete-layers. Because of the separation observed between layers in both tests, it appears desirable to place a tack coat between compacted lifts of asphalt concrete regardless of the time of placement of subsequent lifts.

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