DRAFT

Performance of Drained and Undrained Flexible Pavement Structures in Accelerated Loading under Wet Conditions

Summary Report – Goal 5

Partnered Pavement Performance Program

Prepared for:

California Department of Transportation

by:

Manuel O. Bejarano, John T. Harvey, Abdikarim Ali, David Mahama, David Hung, and Pitipat Preedonant

March 2003

Pavement Research Center
Institute of Transportation Studies
University of California at Berkeley

TABLE OF CONTENTS

Table of Contents iii

List of Figures v

List of Tables vi

Executive Summary vii

1.0 Introduction 1

2.0 Caltrans experience with ATPB Layers 1

3.0 ATPB Layer Design 3

4.0 ATPB Cost 5

5.0 Recommendations from Accelerated Testing under Dry Conditions 5

6.0 Objectives 7

7.0 HVS Experiment 7

7.1 Pavement Structures 8

7.2 Pavement Materials 10

7.2.1 Wearing Course Mixes 10

7.2.2 Asphalt Concrete Mix 10

7.2.3 ATPB, Base, and Subbase 10

7.3 Environmental Conditions 11

7.4 HVS Test Program 14

7.4.1 Traffic Loading 14

7.4.2 Failure Criteria 15

7.4.3 Pavement Instrumentation and Methods of Monitoring 15

8.0 Summary of HVS Data 15

8.1 Permanent Deformation 16

8.2 Extracted Cores and Visual Observation ... 20
8.3 Percolation Tests ... 22
8.4 Elastic Deflections ... 23
8.5 Crack Length Progression ... 27
8.6 Other Activities ... 30
8.6.1 Air-Void Content ... 30
8.6.2 Dynamic Cone Penetrometer (DCP) ... 31
8.6.3 Open Test Pits ... 32
9.0 Comparison of Pavement Performance ... 32
9.1 Drained versus Undrained Section Performance ... 32
9.2 ARHM versus DGAC Wearing Course Performance ... 33
9.3 Dry versus Wet Condition Performance ... 33
10.0 Conclusions ... 34
11.0 Recommendations ... 35
12.0 References ... 36

LIST OF FIGURES

Figure 1. Typical cross section of flexible pavement with ATPB layer.(*1*) 2

Figure 2. Layout of the pavement test sections at the Pavement Research Center Accelerated Pavement Test Facility located at the University of California at Berkeley Richmond Field Station. 9

Figure 3. Schematic illustration of the water drip system. 12

Figure 4. Photograph of the water drip system. 13

Figure 5. Summary of accumulated average maximum rut depths. 17

Figure 6. Summary of in-depth permanent deformations. 19

Figure 7. Core from Section 543, station 13 along the centerline (trafficked area) showing stripped ATPB separated from core. 21

Figure 8. Core from Section 543, station 13 outside the trafficked area showing ATPB still intact. 22

Figure 9. Summary of percolation tests. 23

Figure 10. Summary of road surface deflections measured with the RSD under a 40-kN load. . 24

Figure 11. Summary of estimated elastic deflection in pavement layers based on a 40-kN HVS load. 26

Figure 12. Summary of crack length accumulation. 29

Figure 13. Summary of air-void contents in wearing courses and ATPB layer. 30

LIST OF TABLES

Table 1 Test Sections for Goal 5 Experiment 7

Table 2 Design and As-Built Layer Thicknesses 8

Table 3 Summary of Load Applications and ESALs 15

Table 4 Pavement Instrumentation 16

Table 5 Contribution of Pavement Layers to Surface Rutting Based on MDD Measurements 20

Table 6 Contribution of Pavement Layers to Surface Elastic Deflection Based on MDD Measurements for a Test Load of 40 kN 27

Table 7 HWD Deflections Normalized to 40-kN Load, Before and After HVS Testing 28

Table 8 DCP Penetration Rates for Unbound Layers 31

Table 9 Estimated ESALs (Millions) for Failure Criteria 32

Table 10 Estimated ESALs (Millions) for Failure Criteria for Sections Under Dry Experiment 34

EXECUTIVE SUMMARY

The California Department of Transportation (Caltrans) requires that all new flexible pavements include a 75-mm layer of asphalt treated permeable base (ATPB) between the asphalt concrete and the aggregate base layers. The purpose of the ATPB layer is to intercept water entering the pavement because of high permeability resulting from insufficient compaction or through cracks in the asphalt concrete layer and transport it away from the pavement before it reaches the unbound materials.

This reports summarizes the results of a study using Heavy Vehicle Simulator (HVS) trafficking to evaluate the performance of drained and undrained flexible pavements under wet (saturated base) conditions. A drained structure is a pavement section that contains an ATPB layer between the asphalt concrete and the aggregate base. An undrained structure is a pavement section that does not contain an ATPB layer. Wet conditions used in this study were intended to simulate approximate surface infiltration rates for a badly cracked asphalt concrete layer that would occur along the northwest coast of California during a wet month.

This program included HVS tests on three intact sections that remained after completion of the Goal 2 and Goal 3 studies.* The test sections consisted of the following:

Test Section Number	Pavement Type	Wearing Course Type
543RF	Drained (with ATPB)	40 mm ARHM-GG
544RF	Undrained (no ATPB)	40 mm ARHM-GG
545RF	Undrained (no ATPB)	75 mm DGAC

Results of the accelerated loading included the following:

* The Goal 1 study encompassed accelerated loading on four full-scale pavements with untreated aggregate base and asphalt treated permeable base. For the Goal 3 study, accelerated loading was applied to overlays of the four test sections that had been cracked in the Goal 1 tests. The overlays consisted of conventional dense-graded asphalt concrete (DGAC) and asphalt rubber gap graded hot mix (ARHM-GG).

1. The ATPB placed between the asphalt concrete and aggregate base *stripped* (the adhesive bond between the aggregate surface and the asphalt binder was broken) in the presence of water and heavy loading.
2. Clogging of the ATPB with fines from the aggregate base was observed in the wheel path area. This resulted in a reduction in permeability of the ATPB by three orders of magnitude (10 mm/s. To 0.01 mm/s.) which contributed to a saturated condition and accelerated its deterioration.
3. While the ATPB initially reduced surface deflections under load, rapid deterioration of the ATPB with load repetitions resulted in similar deflections to those observed in the undrained sections.
4. When the ATPB stripped, an increase in permanent deformation was observed as compared to the dry conditions.
5. Because of the stripping of the ATPB, surface rutting was the prominent failure mode; on the other hand, for the undrained sections, fatigue cracking was the predominant failure mode.

Based on this test program as well as the tests and analyses associated with the Goal 1 and 3 studies, the following recommendations are made:

1. Improved compaction of the asphalt concrete layer and the use of improved structural section design (to minimize fatigue cracking in the asphalt concrete), both of which would substantially reduce the ingress of water into the pavement, suggest that consideration should be given to the elimination of the ATPB directly beneath the asphalt concrete layer.

2. Because of the susceptibility of ATPB as currently specified to the action of water, improved mix design is recommended using more asphalt and/or modified binders such as asphalt rubber.
3. If ATPB is used, suitable soil or fabric filters should be incorporated in the structural pavement sections.

1.0 INTRODUCTION

The California Department of Transportation (Caltrans) Highway Design Manual (HDM) requires that a highly permeable drainage layer consisting of 75 mm of asphalt treated permeable base (ATPB) should be placed immediately below the asphalt concrete layer to intercept surface water that enters the structural section.(*1*) Exceptions can be justified for areas where the mean annual rainfall is very low (< 125 mm) or where the subgrade soil is free-draining (permeability > 0.35 mm/s). Figure 1 illustrates the location of the ATPB in a typical flexible pavement section. This layer extends laterally from 0.3 m outside the edge of the traveled way on the high side to the edge of the collector trench on the low side. Figure 1 shows the ATPB layer in a typical flexible pavement section. In situations in which there is concern that the infiltrating surface water may saturate and soften the underlying unbound layers, a prime coat or other membrane is applied to the surface of the unbound layer to prevent erosion. The permeable base layer is then placed on top of the unbound layer. A separate subsurface drainage system is usually designed to intercept groundwater flows.

2.0 CALTRANS EXPERIENCE WITH ATPB LAYERS

Caltrans has experimented with asphalt treated permeable materials for use as drainage layers for at least 35 years and has used ATPB as a standard component in new pavement designs since 1983. By the early 1990's, Caltrans personnel suspected that ATPB materials in some projects were experiencing moisture damage in the form of stripping of the asphalt binder from the aggregate surface.[*] These projects included both asphalt concrete and portland cement

[*] Stripping is an advanced state of moisture damage in a mix from debonding due to adhesive failure between the asphalt binder film and aggregate surface.

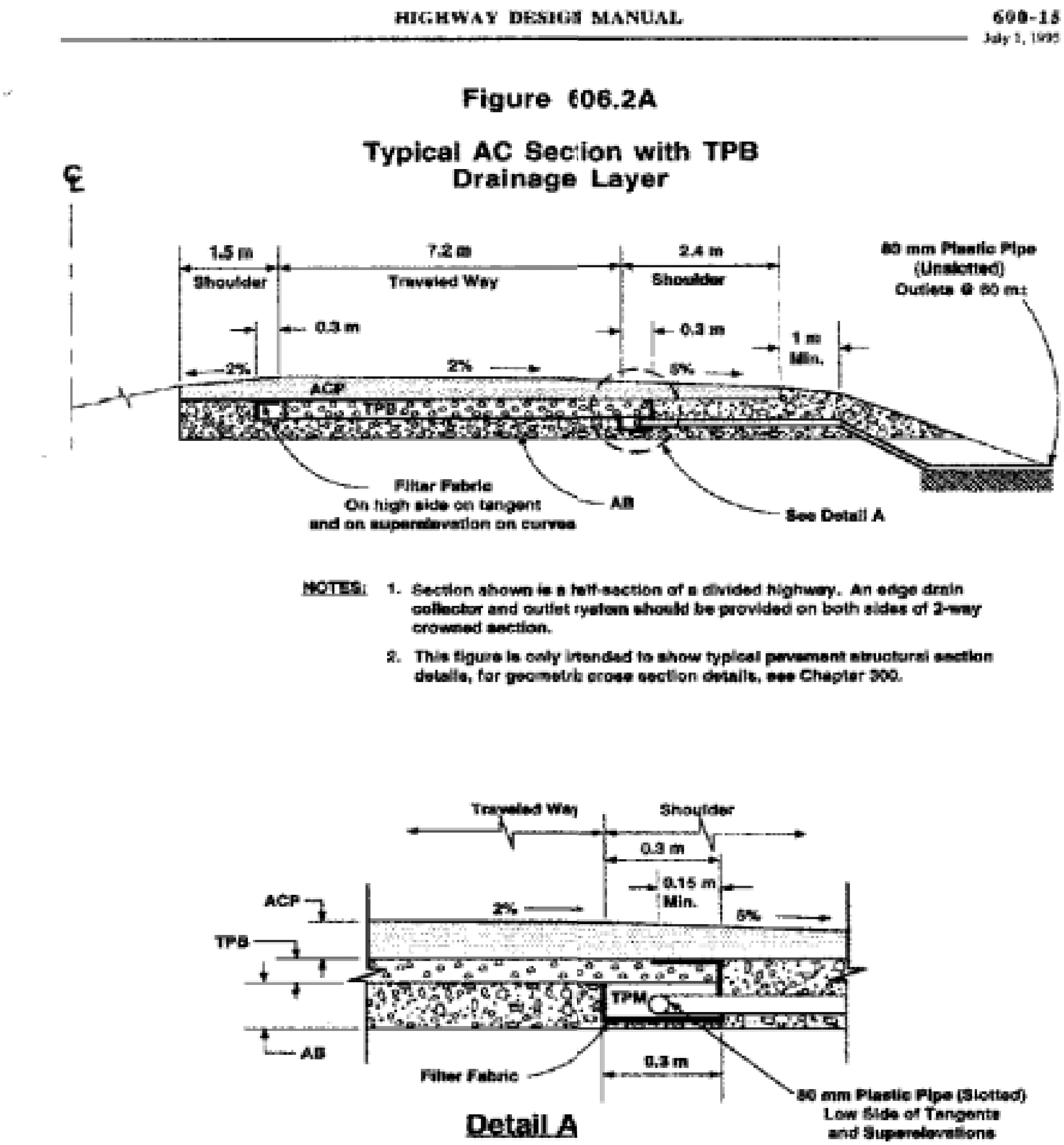

Figure 1. Typical cross section of flexible pavement with ATPB layer.(*1*)

concrete pavements. Stripping contributes to faulting in portland cement concrete pavements and loss of structural strength in asphalt concrete pavements.

An investigation of nine in-service pavement sections containing ATPB indicated that the ATPB was either stripped or that intrusion of fines was present in the ATPB layer.(*2*) This investigation concluded that stripping of the ATPB within 10 years of construction is not uncommon in asphalt concrete surfaced pavements. Stripping may be common and progress rapidly at locations where large quantities of water enter the ATPB layer from the surface, such as at the joints of portland cement concrete pavements and at cracks in asphalt concrete pavements, or when an asphalt concrete surface is very pervious due to poor compaction. In those pavements where stripping was observed, it had often proceeded to the point at which no asphalt was found on the aggregate particles or anywhere else in the ATPB layer. The studies indicated that intrusion of fines also occurred and likely reduced the permeability of the ATPB. Other agencies, e.g., the Indiana Department of Transportation and the Ontario Ministry of Transportation, have reported similar observations.(*3, 4*)

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. The current trend is for reduced maintenance funding and staffing, which has resulted in diminishing ability to regularly maintain the drainage systems. This trend is expected to continue. These observations stress the importance of examining ATPB in a soaked state.(*5*)

3.0 ATPB LAYER DESIGN

Caltrans currently uses one thickness and one type of asphalt treated permeable material in its pavement design. The material is fairly uniform with 80 to 100 percent of the aggregate particles between the 9.5-mm and 19-mm sieve sizes. At the time the material was adopted for use, it was required that 25 percent of the particles be crushed. The asphalt content was typically specified to be 1.5 percent by mass of aggregate; AR-4000 asphalt was used. Measured permeability was about 4,575 m/day (53 mm/s). These requirements were similar to those for the 19-mm material proposed by Lovering and Cedergren (*6*). The primary differences between Lovering and Cedergren's recommendations and Caltrans are the asphalt content (2 to 3 percent for Lovering and Cedergren versus 1.5 percent for Caltrans) and aggregate characteristics (50 percent crushed for Lovering and Cedergren versus 25 percent for Caltrans).

Excessive deformations experienced during construction compaction led to changes to the Caltrans specification in 1984. These changes included an increase to 90 percent crushed particles and the use of stiffer asphalt (AR-8000 in place of AR-4000). In addition, the asphalt content was increased from 1.5 to the range 2.0 to 2.5 percent. The specification was further amended in 1987 to increase the stiffness of the binder during the mixing process in order to reduce draindown and eliminate instability encountered in the ATPB after placement. This amendment required that the asphalt be added to the mix when the aggregate temperature is between 135°C and 163°C.

For situations in which fines were present in the unbound soil layers in amounts sufficient to risk clogging of the ATPB, Lovering and Cedergren recommended the use of an underlying base material with a gradation that would permit it to serve as a filter. Current Caltrans practice is based on the assumption that the Class 2 aggregate base and prime coat applied at the surface of the base serves in this capacity.

4.0 ATPB COST

Between 1994 and 1998, the cost of ATPB averaged $56 per cubic meter for an average annual production of 81 thousand cubic meters. For the same period, the cost of aggregate base averaged $26 per cubic meter for 977 thousand cubic meters per year. Thus the cost of ATPB is about 2.1 times that of the aggregate base.

The most cost-effective structure (drained or undrained) would depend not only on the cost difference of constructing both structures, but also on their respective long-term performances. The effectiveness of the ATPB layer in protecting the unbound layers from water infiltration has not yet been evaluated.

5.0 RECOMMENDATIONS FROM ACCELERATED TESTING UNDER DRY CONDITIONS

Accelerated pavement testing (APT) using the Heavy Vehicle Simulator (HVS) of drained and undrained pavement sections under dry conditions (*7–11*) indicated that:

1. Performance of the Caltrans drained and undrained pavement structures under controlled conditions and HVS loading is different.
2. The ATPB layer (drained pavement structure) increases pavement life by increasing the pavement's resistance to fatigue cracking failure.
3. Rutting performance in the unbound layers appears to be similar for both drained and undrained pavement structures.
4. Surface deflections are similar for both drained and undrained pavement structures.

This summary indicates that if it is possible to maintain the structural capacity of the ATPB during the expected life of the pavement, then pavement structures containing ATPB will exhibit better performance (as measured by resistance to cracking) than sections containing only untreated aggregate base. However, if the ATPB is sensitive to moisture, it may lose some of its structural capacity, particularly under heavy traffic loading. Thus one can expect variation in performance of in-service drained pavements depending on the environment, moisture sensitivity characteristics of the ATPB, and maintenance practices.(*11*)

Some studies have provided evidence of this tendency for ATPB to have its structural response characteristics reduced by a combination of temperature and environment.(*2–4*) Thus it was considered extremely desirable to perform the study described herein: to compare the relative performance of *drained* and *undrained* pavements subjected to HVS loading in the "wet" condition.

Based on these test results, it is apparent that the ATPB layer improves the fatigue performance of the asphalt concrete layers in the pavement test sections tested under dry conditions. If the structural capacity of the ATPB is maintained during the life of the pavement, then the pavement structures containing the ATPB will have better fatigue performance than those without an ATPB layer. However, evidence that the ATPB has a tendency to strip and lose its structural capacity when saturated suggests that there may be a great deal of variance in the performance of the drained pavement sections in the field, depending on the environment, stripping potential of the ATPB mix, and maintenance practices.(*12*) Therefore, a study was conducted to compare the relative performance of drained and undrained pavement sections under wet conditions under the Heavy Vehicle Simulator.

6.0 OBJECTIVES

The objectives of this accelerated pavement testing study with the HVS are to:

1. Measure and compare the long-term performance[*] of the drained and undrained pavement structures under conditions in which the aggregate base is exposed to ingress of water from the surface (wet conditions).
2. Measure the effectiveness of the drained pavement in mitigating surface water infiltration and thereby preventing a decrease in stiffness and strength of the unbound layers.
3. Measure and compare the performance of undrained structures with asphalt rubber hot mix gap graded (ARHM-GG) and dense graded asphalt concrete (DGAC) wearing courses under wet conditions as defined in Objective 1 above.

This summary report provides an overall assessment of the results of the three tests conducted to achieve these three objectives. Details of each are included in References (*13–15*)

7.0 HVS EXPERIMENT

Three intact sections were available for this experiment. Table 1 shows the matrix of primary experiment variables, type of pavement section, and associated test section numbers.

Table 1 Test Sections for Goal 5 Experiment

Test Number	Pavement Type	Wearing Course Type
543	Drained (with ATPB)	40 mm ARHM-GG
544	Undrained (no ATPB)	40 mm ARHM-GG
545	Undrained (no ATPB)	75 mm DGAC

[*] Long-term performance of the test sections measured in terms of fatigue cracking and surface rutting.

Sections 543 and 544 are similar except for the presence of the ATPB layer in Section 543. The performances of these two sections have been used to measure the effectiveness of the drainage layer. Sections 544 and 545 permitted evaluation of compactive performance of the two wearing courses. Figure 2 shows the layout of the pavement sections in the Pavement Research Center Accelerated Pavement Test Facility at the University of California, Richmond Field Station.

7.1 Pavement Structures

Cross sections for the three pavement structures are summarized in Table 2, which includes design and as-built thicknesses. For the Goal 1 tests, the layer thicknesses beneath the wearing courses were designed according to the Caltrans design method (*1*) for a subgrade R-value of 10 and a Traffic Index (TI) of 9 (1 million ESALs).(*16*) The term *wearing course* refers to the overlays designed for the Goal 3 tests. Thicknesses were based on the Caltrans overlay design procedure.(*12*)

Table 2 Design and As-Built Layer Thicknesses

Pavement Layer	**Layer Thickness, mm**					
	543-ARHM		**544-ARHM**		**545-DGAC**	
	Design	**As-built**	**Design**	**As-built**	**Design**	**As-built**
Wearing Course	40	36	40	51	75	90
Asphalt Concrete	148	140	162	149	150	143
ATPB	76	64	n/a	n/a	n/a	n/a
Aggregate Base	182	180	274	272	280	259
Aggregate Subbase	215	223	305	205–310	240	206–280
Total	661	643	781	677–782	745	698–772

n/a: ATPB layer not present in these test sections.

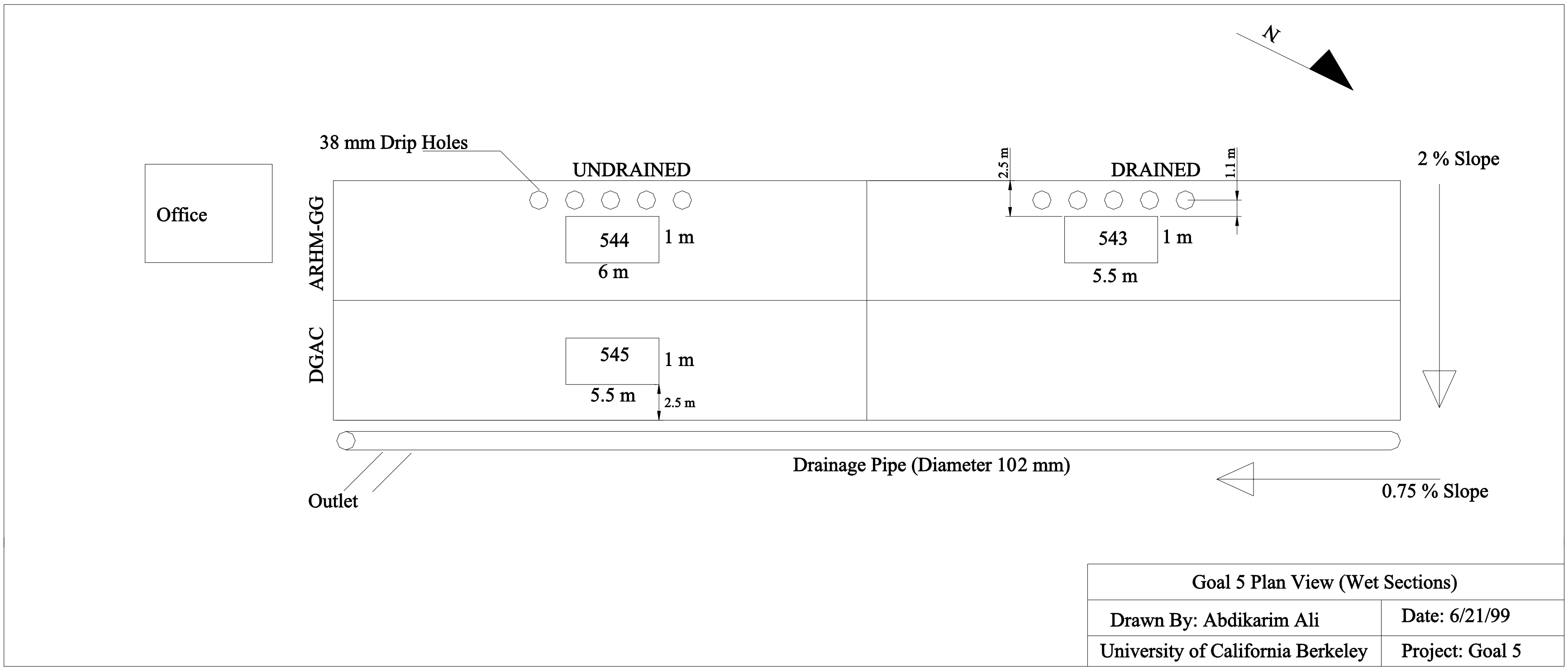

Figure 2. Layout of the pavement test sections at the Pavement Research Center Accelerated Pavement Test Facility located at the University of California at Berkeley Richmond Field Station.

7.2 Pavement Materials

A brief description of the pavement components is included in the following sections. More detailed information is included in References (*12, 16*).

7.2.1 Wearing Course Mixes

The Section 545 DGAC wearing course met Caltrans Standard Specification No. 39 for Type A Asphalt Concrete with a 19-mm maximum coarse grading.

Wearing courses for Section 543 and 544 ARHM-GG met Caltrans Standard Special Provisions for a Type 2 12.5-mm gap-graded asphalt rubber hot mix.

The Performance Grade classifications for the DGAC and ARHM-GG binders are PG 64-16 and PG 82-28, respectively. The extracted binder contents were 5.3 (DGAC) and 6.9 to 7.2 (ARHM) percent by mass of aggregate. Initial in-place compacted air-void contents for the wearing courses ranged from 5 to 7 percent for DGAC and 11 to 15 percent for ARHM.

7.2.2 Asphalt Concrete Mix

The asphalt concrete mix met Caltrans Standard Specification 39 for Type A Asphalt Concrete with 19-mm maximum, coarse grading. The asphalt binder met Caltrans requirements for AR-4000. Binder content was 4.9 percent by mass of aggregate. In-place air-void contents averaged 6.3 and 2.8 percent in the top and bottom lifts, respectively.

7.2.3 ATPB, Base, and Subbase

The asphalt treated permeable base (ATPB), the Class 2 aggregate base (AB), and the Class 2 aggregate subbase (ASB) met Caltrans specifications for materials and compaction. R-values for the AB were between 78 and 83. R-values for the ASB were between 55 and 82.

The subgrade soil for all sections is a high plasticity clay with a liquid limit ranging between 39 and 55 and a plasticity index ranging between 27 and 41. The subgrade classifies as CH and A-7-6 in the Unified Classification System and AASHTO classifications, respectively.

7.3 Environmental Conditions

The test sections were tested under wet conditions and moderate pavements temperatures in an enclosed building (the same building in which the Goal 1 and Goal 3 tests were conducted). A water infiltration system was used to simulate field conditions in which pavement cracks allow water to enter into the pavement. The system drips water into the base through small holes drilled in the asphalt concrete. The holes were drilled at 0.5-m intervals outside and up-slope of the test sections. The drip system consists of a twelve-meter long PVC pipe with twenty-five emitters attached at the end of the dripping line. Each drip system has a water filter that collects any dirt and deposits from the water line together with an electronic valve and timer to control the amount of flow. The programmable drip system was run automatically 24 hours a day, seven days a week. Figure 3 presents a schematic of the water drip system. Figure 4 presents a photograph of the system in operation.

The design rate of water infiltration into the base was maintained at a quantity of 61.5 gallons per day over an area of 7.4 m wide by 12 m long. This rate of water infiltration simulates an average peak week precipitation in Eureka, CA of 51.3 mm. Adjustments were made for the undrained sections because the rate of water inflow exceeded the rate of water infiltration into the aggregate base. For these sections, water inflows were reduced accordingly to approximately 9.5 liters per day (2.5 gallons per day).

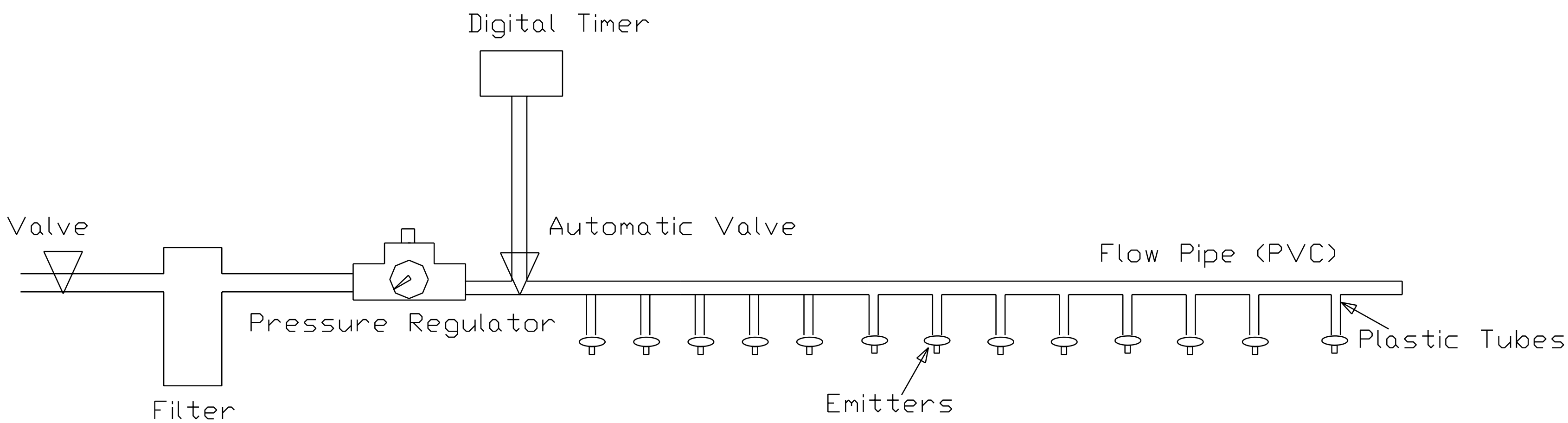

Figure 3. Schematic illustration of the water drip system.

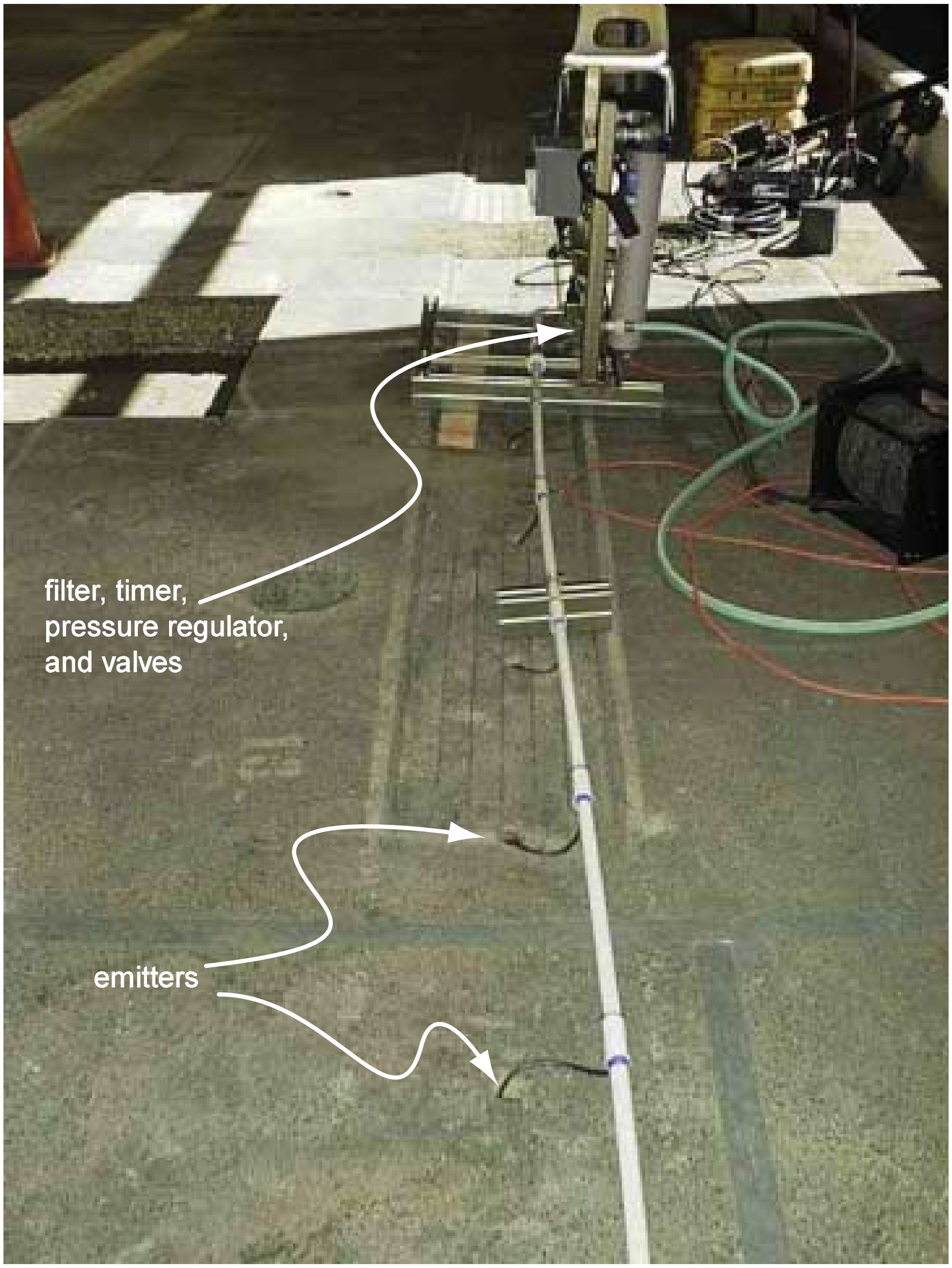

Figure 4. Photograph of the water drip system.

The subgrade has access to the ground water table, which resulted in changes in the water contents of the unbound materials depending on the water table depth. The water table is typically located approximately 3 to 5 meters below the surface of the pavement and fluctuates seasonally.

The target temperature for each test section was 20ºC at a pavement depth of 50 mm. Trafficking was stopped if the surface temperature varied by more than 2ºC of the target value.

7.4 HVS Test Program

Details of the HVS program for each test section are presented in Reports (*13*), (*14*), and (*15*) for Sections 543, 544, and 545, respectively. Only a brief summary of the test programs is included herein.

7.4.1 Traffic Loading

Each HVS test section was 8 m in length by 1 m wide. All test sections were trafficked in the bi-directional mode. Wheel speed was approximately 7.5 km/hr in one direction and 6.8 km/hr in the other. All trafficking had a wander pattern distributed across the 1-m width of the test sections. Pavement performance in the one-meter turn around areas at the end of the trafficked sections was not included in the performance evaluations. Dual radial tires on 11-cm wide rims were used at a 720-kPa inflation pressure. Loading followed the sequence utilized in previous HVS tests starting with 40 kN, increased to 80 kN and then to 100 kN. Table 3 summarizes the load sequence for each load for the three test sections.

Table 3 Summary of Load Applications and ESALs

Test Section	HVS Load Applications × 10^3			ESALs × 10^6
	40 kN	80 kN	100 kN	
543 ARHM-GG/Drained	236.6	331.8	629.3	35.8
544 ARHM-GG/Undrained	176.3	210.7	718.1	37.7
545 DGAC/Undrained	147.0	236.7	358.2	21.3

7.4.2 Failure Criteria

Failure criteria used to define the limiting number of repetitions are as follows:

- Cracking density of 2.5 m/m^2 or more, and
- Maximum surface rut depth of 12.5 mm or more.

7.4.3 Pavement Instrumentation and Methods of Monitoring

Pavement monitoring instrumentation used in the test sections are listed in Table 4 and is described in detail in Reference (*16*).

8.0 SUMMARY OF HVS DATA

As noted earlier, Table 3 summarizes the sequence of loading, number of load applications for each trafficking load, and equivalent number of ESALs (assuming the load equivalency factor exponent of 4.2 used by Caltrans) applied to each test section.

Upon completion of the HVS tests, performance of each section as measured by permanent deformation, elastic deformation, and fatigue cracking at the pavement surface was evaluated. Results are included in the following sections.

Table 4 Pavement Instrumentation

Equipment	Measured Parameter(s)
Multi-Depth Deflectometer (MDD)	Elastic vertical deflections and permanent vertical deformations at various levels in the pavement structure, relative to a reference depth in the subgrade (four MDDs were included in each section)
Road Surface Deflectometer (RSD)	elastic vertical deflections at the surface of the pavement
Falling Weight Deflectometer (FWD)	elastic vertical deflections at the surface of the pavement
Thermocouples	temperatures at various depths in the asphalt bound materials
Laser Profilometer and Straight Edge	transverse profile of the pavement surface to determine surface rutting
Dynamic Cone Penetrometer (DCP)	relative shear resistance of unbound layers
Digital Image Analysis of Cracking	extent of surface cracking;(*11*)
Nuclear Density Gauge	density and water content of the unbound layers at the completion of trafficking, inside and outside the trafficked area
Nuclear Hydro-Probe	water contents in the unbound layers just outside the trafficked area during testing of the section
Trenches and Cores	pavement thicknesses, water contents, and air-void contents of asphalt bound materials inside and outside the trafficked area at the completion of the test; direct observation of the condition of each pavement layer

8.1 Permanent Deformation

Figure 5 shows accumulated average maximum surface rut depths with number of load applications for each level of trafficking. Surface ruts were measured with the laser profilometer transversely every 0.5 meters along the section except for the turnaround zones.

The data show that under the 40-kN load, the initial bedding-in phase of pavement rutting is slightly higher in the section with the DGAC wearing course that the rutting in the sections with the ARHM wearing course. For the 80-kN load, surface rutting is slightly higher (about 2

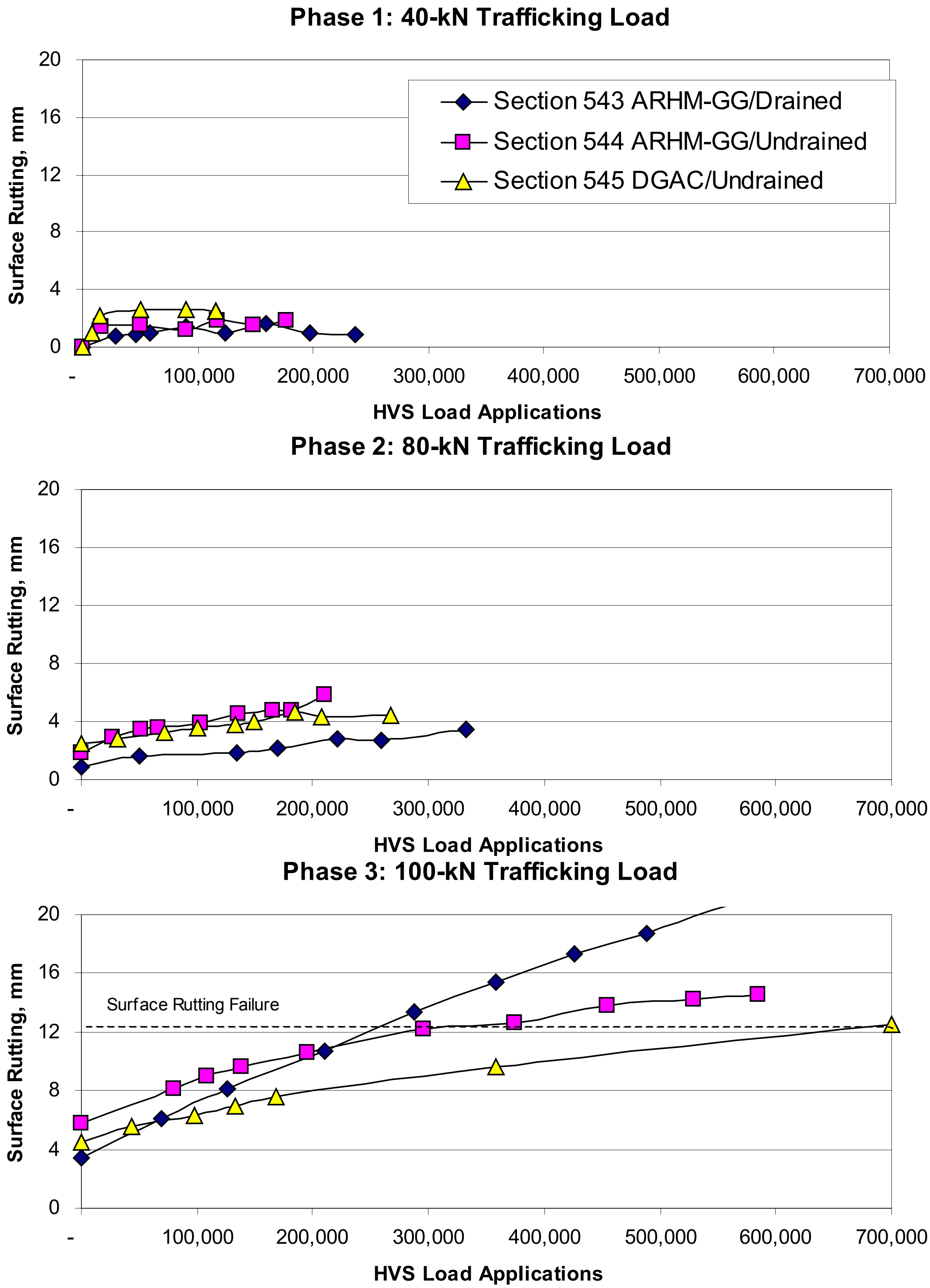

Figure 5. Summary of accumulated average maximum rut depths.

mm) for the undrained sections than for the drained section. For the 100-kN trafficking load, rates of surface rutting accumulation are different for the drained and undrained sections.

The rutting rates for both undrained sections are similar, gradually decreasing with the number of load applications. Section 545 (DGAC wearing course) showed less rutting than Section 544 (ARHM wearing course). This difference results from the difference in the thicknesses of the wearing courses—90 mm for Section 545 versus 38 mm for Section 544. The surface rutting rate of the drained section is about 1.5 times that of the undrained sections. This rapid increase of surface deformation with load applications under the 100-kN load attributed from the weakening of the ATPB layer is due to the presence of water.

Figure 6 shows the development of in-depth permanent deformations with HVS load applications as measured with the MDDs. Contributions of each pavement layer to surface deformation is slightly different among the sections. In Sections 543 and 544, both the bound layers and the aggregate base contributed significantly to the total surface deformation. In Section 545, on the other hand, the major contributor to surface deformation was the aggregate base. Rutting at the surface of the aggregate subbase was less than 5 mm for each of the three sections. Permanent deformation at the subgrade could not be recorded because the MDD at that depth in each of the three sections was inoperable at the time of testing. Based on these measurements, the contribution of the subbase and subgrade layers was estimated to be less than 20 percent of the total rut depth measured at the pavement surface. Table 5 summarizes the approximate contributions of each pavement layer to surface rutting with the MDDs for each level of traffic.

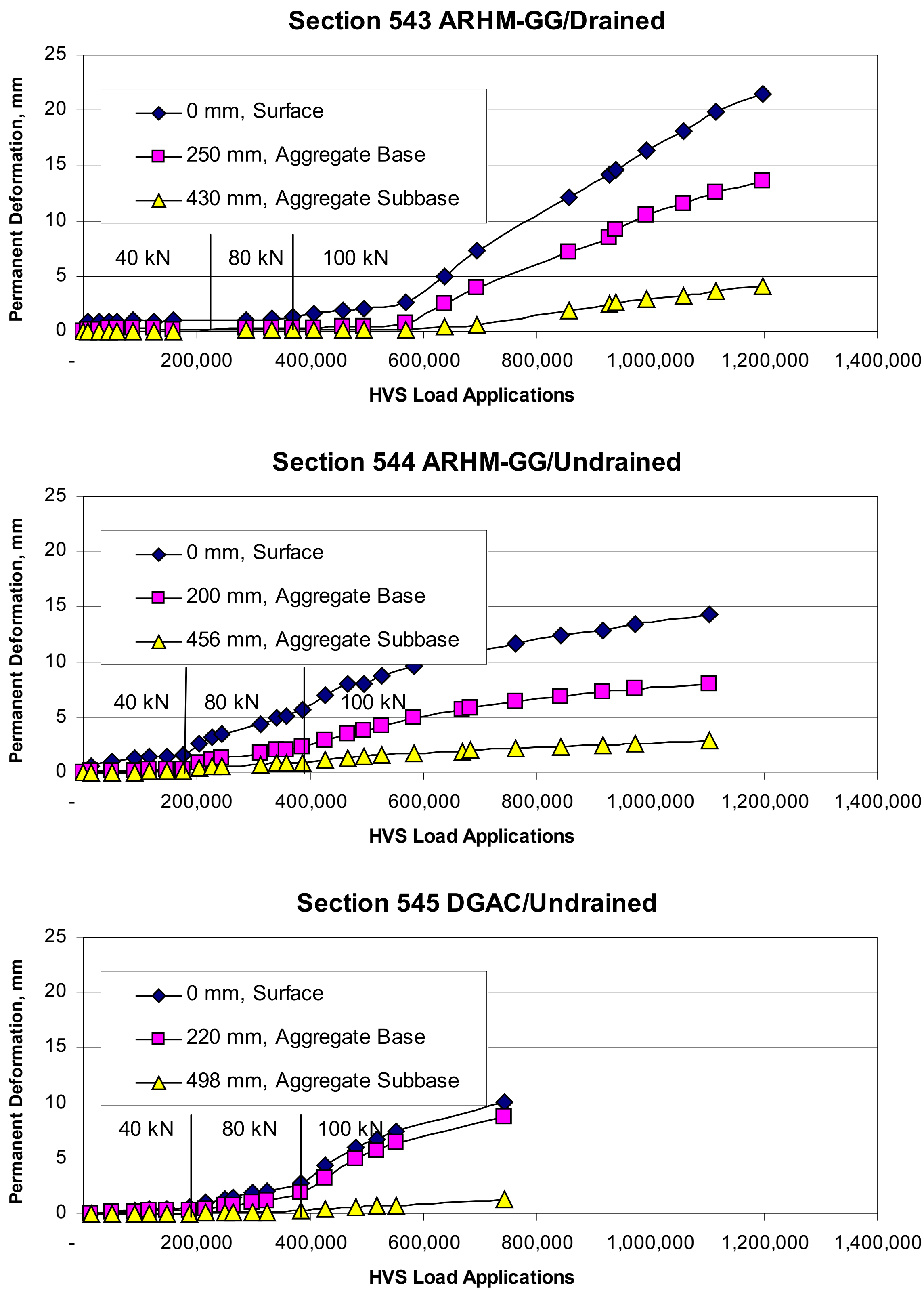

Figure 6. Summary of in-depth permanent deformations.

Table 5 Contribution of Pavement Layers to Surface Rutting Based on MDD Measurements

Layers	Surface Rutting Contribution (percent) by Layer under Specified Load for Each Section								
	40 kN			80 kN			100 kN		
	543	544	545	543	544	545	543	544	545
Bound	75	85	43	77	59	48	38	44	16
Aggregate Base	22	9	44	13	24	40	45	36	74
ASB and Subgrade	2	6	13	10	16	11	18	19	10

Based on the failure criterion of 12.5 mm for limiting maximum surface rutting, the drained section (Section 543) failed at about 850,000 HVS load repetitions, as shown in Figure 5. The undrained section with the ARHM-GG wearing course (Section 544) also failed at about this same number of repetitions. At this number of HVS load repetitions, the undrained section with the DGAC (Section 545) had developed a rut depth of 8 mm.

8.2 Extracted Cores and Visual Observation

Cores obtained from the centerline and outside of the trafficked area in Section 543 show that the ATPB stripped. Those extracted from the center of the trafficked area showed that the ATPB layer could not be extracted as a solid core because the asphalt binder was completely stripped from the aggregate, as illustrated in Figure 7. On the other hand, cores extracted from outside of the trafficked area showed that the ATPB layer was intact with no stripping problems, as shown in Figure 8. Stripping in the ATPB likely resulted from water in the voids being forced through the asphalt films to the aggregate surface by the excess pore water pressure resulting from the HVS loading.

Cores and slabs also showed significant contamination by fines from the aggregate base in the interfaces of the asphalt concrete lifts as well as in the ATPB. A significant number of the cores from the trafficked areas showed no bonding between the asphalt layers. This was

a. Asphalt Layers

b. Asphalt Treated Permeable Base

Figure 7. Core from Section 543, station 13 along the centerline (trafficked area) showing stripped ATPB separated from core.

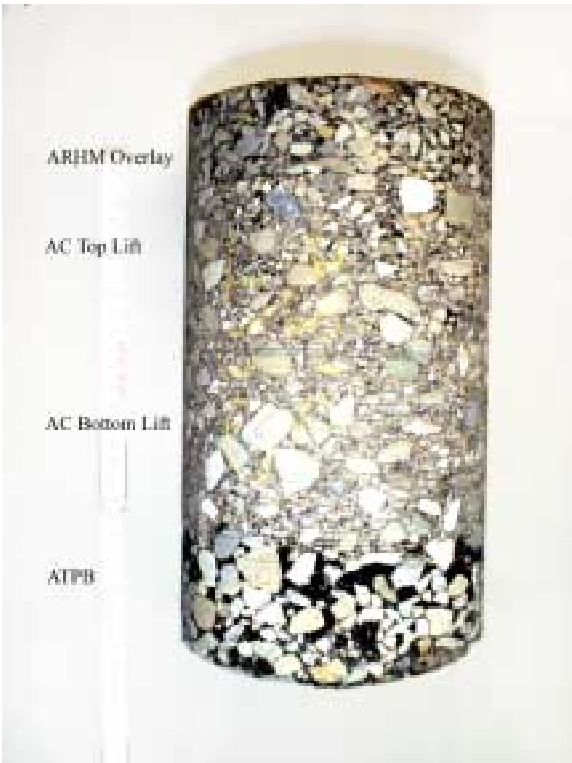

Figure 8. Core from Section 543, station 13 outside the trafficked area showing ATPB still intact.

observed in the earlier HVS tests on the pavement sections tested in the dry condition, reiterating the need for tack coats between lifts, e.g., Reference (*11*).

8.3 Percolation Tests

To further study the failure of the ATPB layer, a series of percolation tests were conducted on drained pavement sections previously tested under the HVS at various locations [Reference (*17*) contains a detailed description of the tests and their locations]. These percolation tests are summarized in Figure 9.

Percolation tests on the ATPB in the untrafficked condition had a permeability of about 1 cm/sec. Tests conducted in the trafficked portion showed a significant reduction in the

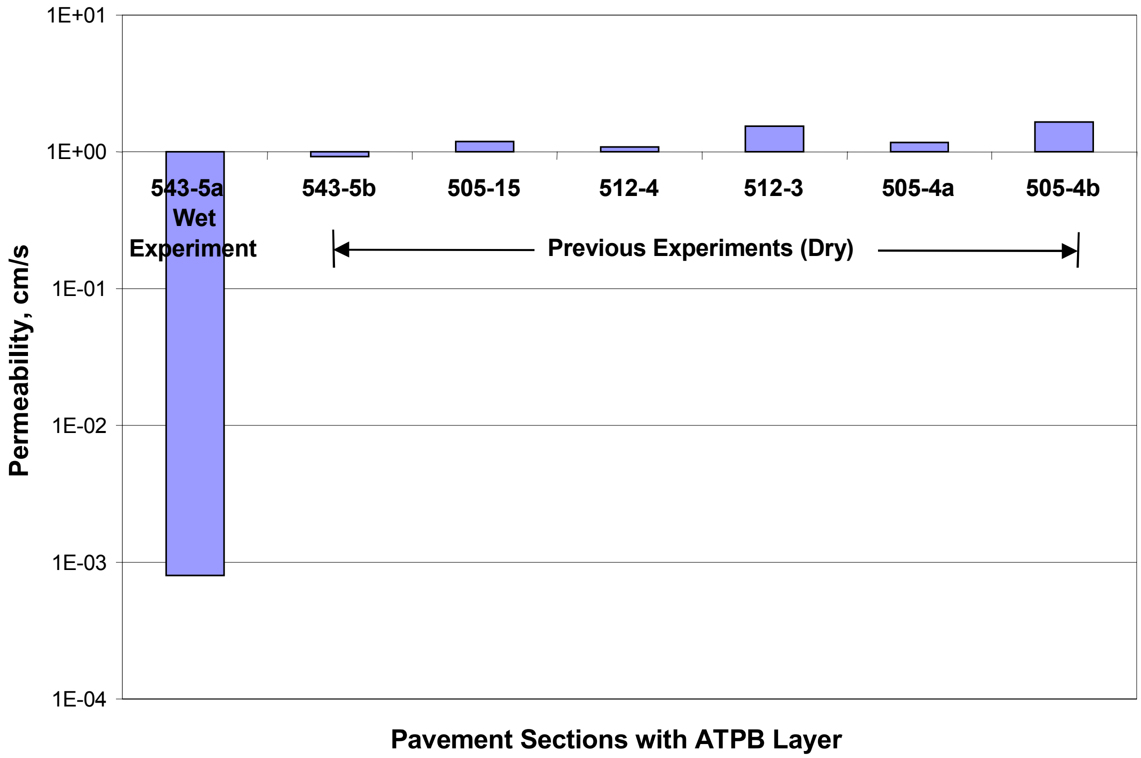

Figure 9. Summary of percolation tests.

permeability the ATPB to about 0.001 cm/sec. Theses data are supported by the observations from the test pit reported in the previous section, which clearly showed the intrusion of fines. The associated significant reduction in permeability of the ATPB contributed to the buildup of pore water pressure and subsequent stripping leading to the reduced structural capacity of the pavement section, e.g., as shown by the increase in deflection for Section 543.

8.4 Elastic Deflections

Average surface elastic deflections measured along the centerline with the RSD under a 40-kN test load are presented in Figure 10. The data show that the RSD elastic deflections of the drained section (Section 543) are 60 to 70 percent lower than those of the undrained sections (Sections 544 and 545) during the 40-kN trafficking stage. During the 80-kN trafficking stage,

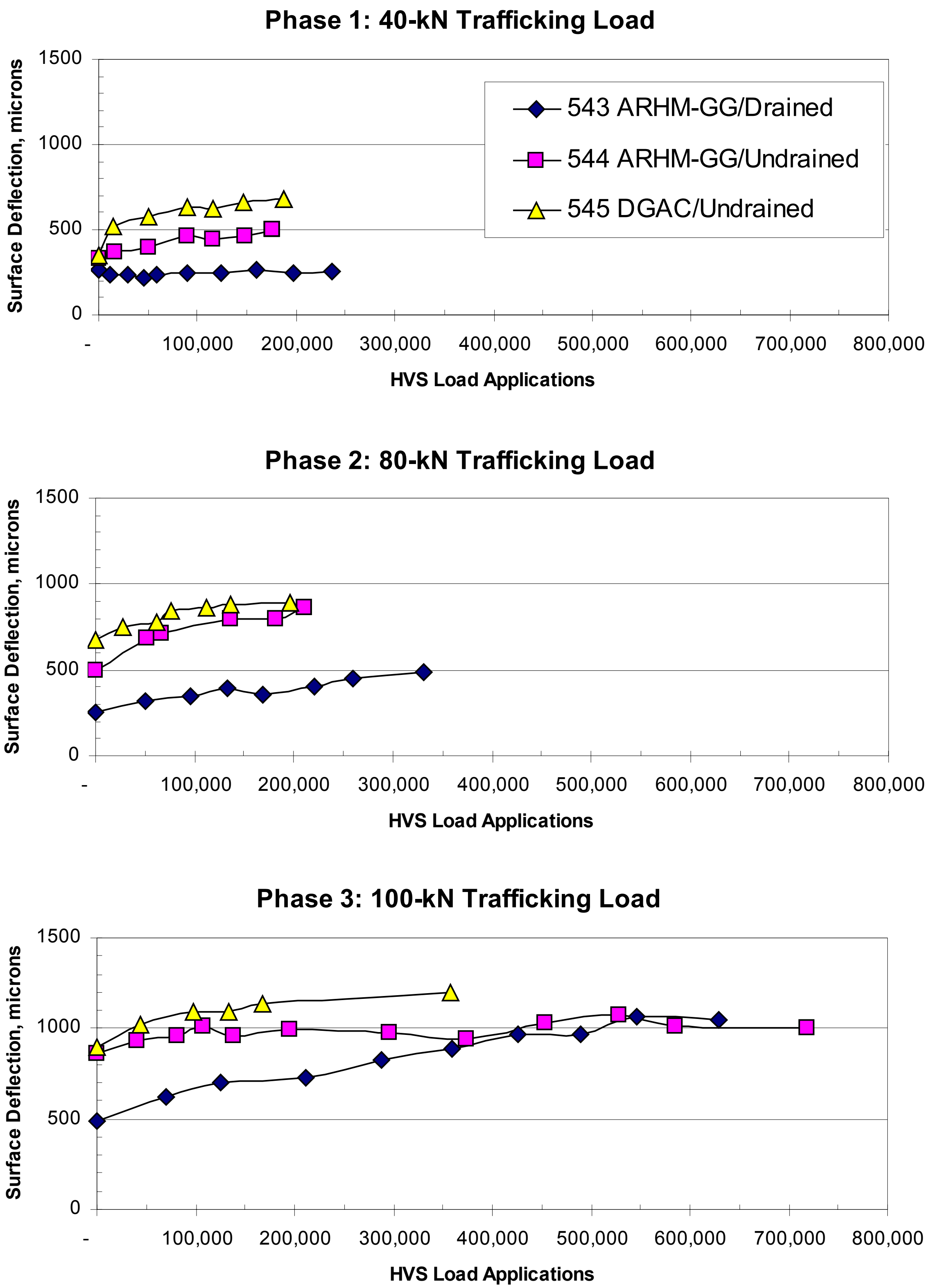

Figure 10. Summary of road surface deflections measured with the RSD under a 40-kN load.

the surface deflections of both undrained sections are similar to one another while the surface deflections of the drained section was approximately 50 percent less than those of the undrained sections. During the 100-kN trafficking stage, the elastic deflections of the drained section rapidly increased with the number of load applications, with the level of elastic deflections on the drained section approaching a similar value to that of the elastic deflections measured on the undrained sections. Stripping of the ATPB likely accounted for the this significant increase.

Figure 11 illustrates calculated values for elastic deflections of the pavement layers determined from in-depth elastic deflections measured with the MDD. This figure shows that the elastic deflections in the drained section remained lower than those of the undrained sections for the 40-and 80-kN trafficking loads. The progressive failure of the ATPB caused an increase in the elastic deflections of the lower unbound layers (subbase and subgrade). However, elastic deflections in the bound and aggregate base layers do not seem to change significantly during the 80- and 100-kN trafficking stages

The contributions of the pavement layers to surface deflections are presented in Table 6. These values were obtained from in-depth elastic deflections measured from the MDD installations. The percent contribution of each of the layers is reasonably consistent throughout the three levels of trafficking for all three sections. A comparison of the contribution of the pavement layers to surface rutting with the contribution of the pavements layers to surface elastic deflection shows an inverse relationship between the two. While the bound layers yielded high contributions to surface rutting, they yielded low contributions to surface elastic deflections. Inversely, the aggregate subbase and the subgrade yielded consistent high contributions to surface elastic deflections while yielding low contributions to surface permanent deformation.

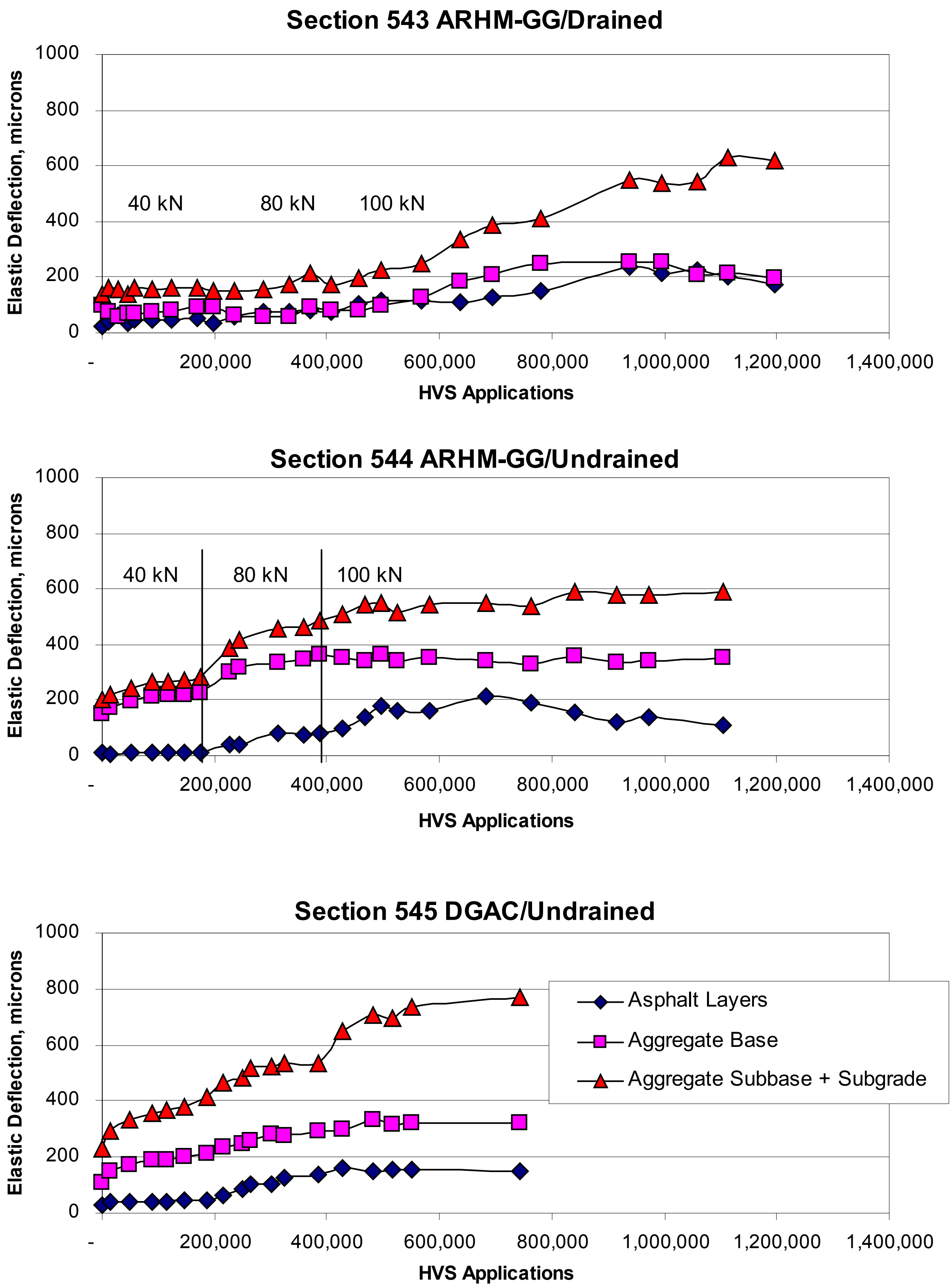

Figure 11. Summary of estimated elastic deflection in pavement layers based on a 40-kN HVS load.

Table 6 **Contribution of Pavement Layers to Surface Elastic Deflection Based on MDD Measurements for a Test Load of 40 kN**

Layers	Surface Elastic Deflection Contribution (percent) by Layer for the Three Stages of Traffic Loading								
	40 kN			80 kN			100 kN		
	543	544	545	543	544	545	543	544	545
Bound	16	2	5	24	7	5	20	14	13
Aggregate Base	29	44	33	22	40	31	25	33	27
ASB and Subgrade	56	54	62	53	53	61	55	53	64

FWD deflection tests were conducted on the test sections before and after HVS testing. A total of 27 tests were conducted on each test section. Each test was performed at three load levels and normalized to a 40-kN load for analysis. Table 7 summarizes deflections measured at the load plate. Before HVS testing, Section 544 (AR/Undrained) showed the highest deflections followed by similar elastic deflections of Section 545 (AC/Undrained) and 543 (AR/Drained). Coefficients of variation for the deflections before HVS testing were less than 11 percent for all sections. The values for Section 544 (AR/undrained) were less than 5 percent.

After HVS testing, coefficients of variation for Section 544 and 545 remained below 11 percent but significantly increased for Section 543 to approximately 40 percent. These results suggest that there may be a significant variability in the structural capacity along this section due in part to stripping in the ATPB.

8.5 Crack Length Progression

Figure 12 shows surface crack lengths measured during the 100-kN load trafficking phase. No surface cracks were observed during the 40- and 80-kN load phases. The observed cracks under the 100-kN trafficking load were predominantly transverse hairline cracks and were sometimes difficult to detect visually. Typical widest crack widths were approximately 0.2 mm. These hairline cracks did not spall or increase significantly in width during HVS trafficking.

Table 7 HWD Deflections Normalized to 40-kN Load, Before and After HVS Testing

Date	543 ARHM-GG Drained			544 ARHM-GG Undrained			545 DGAC Undrained		
	Avg. Normalized Deflections (µm)	Standard Deviation (µm)	Average Pavement Temperature (°C)	Avg. Normalized Deflections (µm)	Standard Deviation (µm)	Average Pavement Temperature (°C)	Avg. Normalized Deflections (µm)	Standard Deviation (µm)	Average Pavement Temperature (°C)
	Before HVS Testing								
July ’99	124	10	18.6	146.7	7.1	18.6	127.5	11.5	19.0
Aug. ’99	130	12		157.0	7.5	21.0	132.6	11.7	19.8
Sept. ’99	129	11	21.4	158.5	6.0	21.4	136.8	13.8	20.2
Oct. ’99	126	11	23.0	161.3	7.5	23.0	138.4	14.5	22.0
Nov. ’99	123	10	21.0	153.6	7.4	21.0	130.1	14.0	18.8
Feb. ’00				142.3	7.0	13.0	116.9	8.8	12.5
Mar. ’00				149.8	7.2	17.0	119.2	7.1	18.0
June ’00							142.9	9.2	23.0
Oct. ’00							129.9	8.7	18.0
	After HVS Testing								
May ’00	347	134	15.0						
June ’00				480.7	18.0	22.0			
Oct. ’00				331.4	19.9	20.0			
July ’01				301.2	11.0	20.0	446.4	34.7	20.0

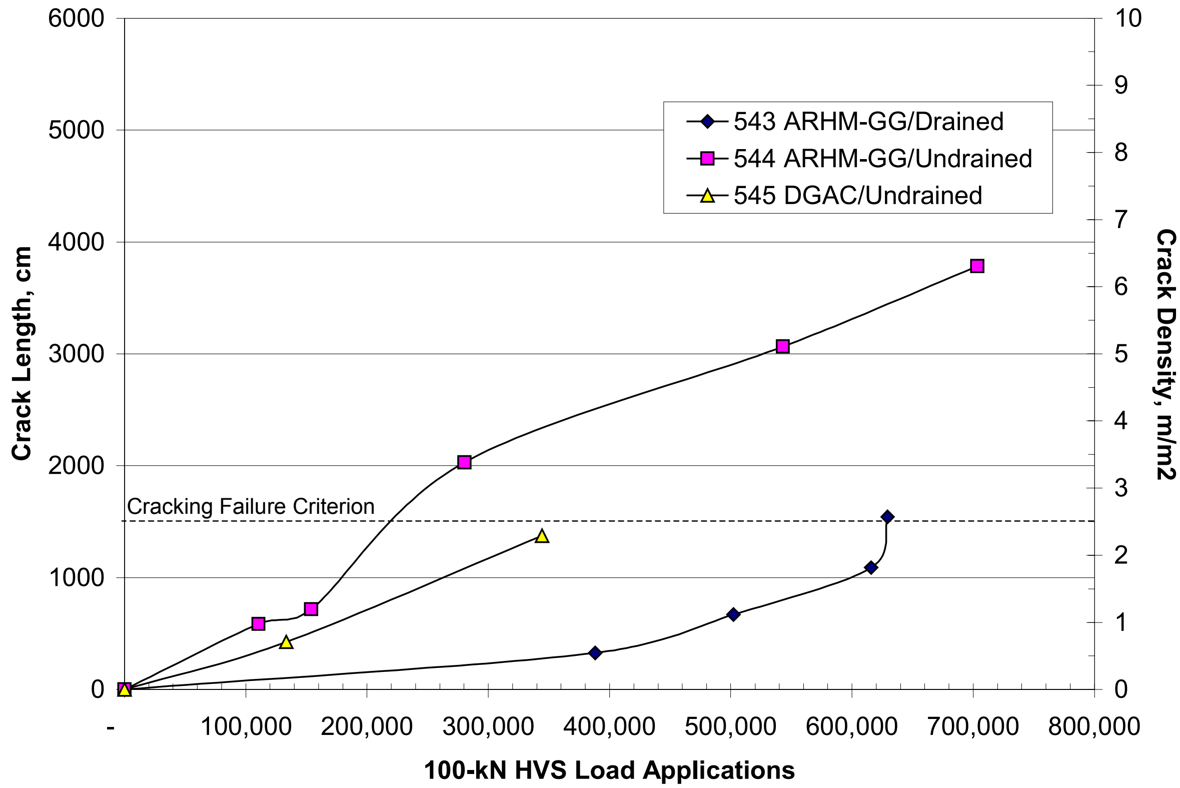

Figure 12. Summary of crack length accumulation.

Inspections of Section 543 (ARHM-GG/drained) during HVS testing revealed deposits of fines accumulated on the pavement surface along some of the cracks. These fines, primarily silty material, were periodically removed but reappeared after some trafficking. It is likely that these fines were pumped from the aggregate base through cracks in the asphalt concrete layers to the surface. Pumping was not observed on the undrained sections.

Figure 12 shows that surface cracks developed faster in the undrained sections than in the drained section. Using the failure criterion of 2.5 m/m^2 of surface cracks, the drained section sustained about 300,000 to 450,000 more load applications of the 100-kN trafficking load than both undrained sections. This is in spite of the stripping in the ATPB. (It should be noted however as seen in Figure 4 that the limiting rut depth was reached at about 250,000 HVS load applications.

8.6 Other Activities

Core sampling of the bound layers, dynamic cone penetrometer testing, and trenching of the test sections were included as post-trafficking activities in the HVS test program. A summary of these activities is included in the following sections.

8.6.1 Air-Void Content

Air-void content data obtained from the extracted cores are presented in Figure 13. The figure shows data for the wearing courses on all three sections and for the ATPB layer of Section 543. A significant reduction in air-void content is observed for the ARHM-GG wearing courses along the centerline of Sections 543 and 544. The poor compaction and, therefore, high initial air-void content in the ARHM-GG wearing courses were anticipated and presented in other

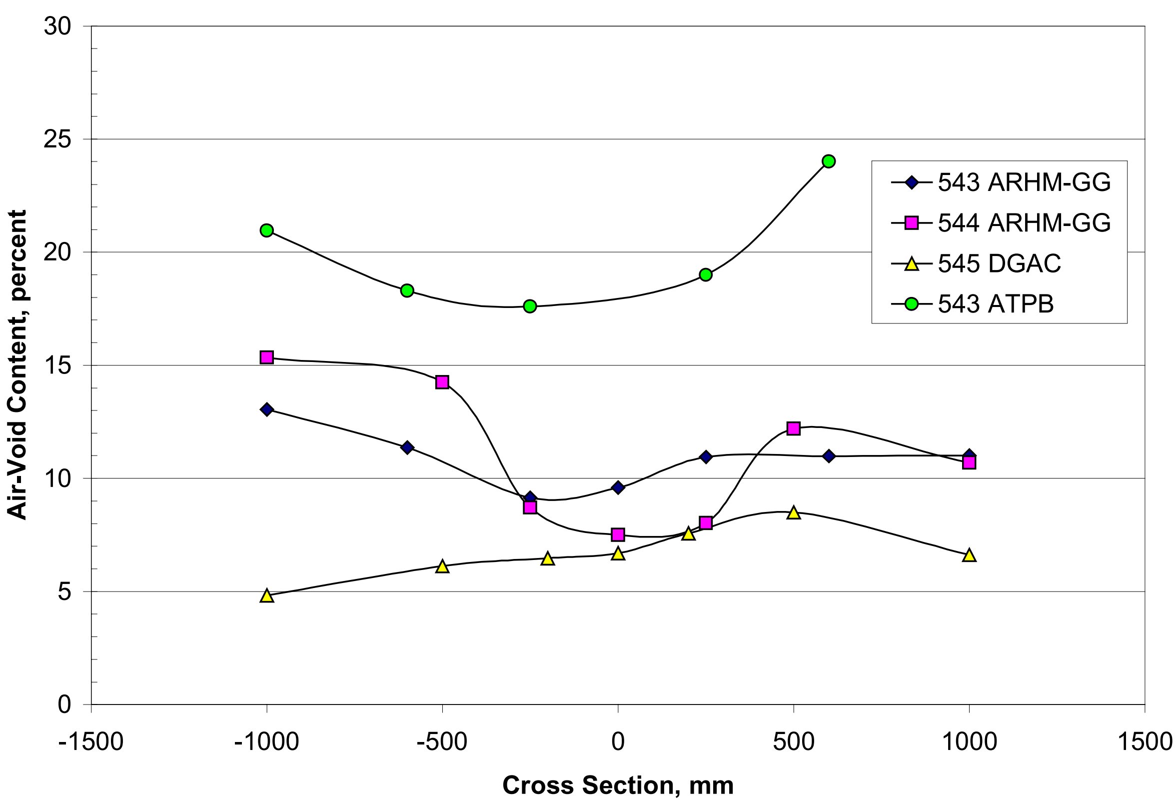

Figure 13. Summary of air-void contents in wearing courses and ATPB layer.

reports.(*12, 16*) Reduction in air-void content along the centerline is not evident for the DGAC wearing course in Section 545.

The data also show that lower initial air-void contents were obtained in the non-trafficked areas at –1000 mm from the centerline. Better compaction along this part of the sections was obtained because of the presence of a k-rail wall, which provided lateral support and confinement of the mix during the compaction.

8.6.2 Dynamic Cone Penetrometer (DCP)

DCP tests were conducted along centerline (trafficked) and non-trafficked areas. For the undrained section, average centerline penetration rates (mm per blow) were 10 to 33 percent lower than those from the non-trafficked areas. The apparent decrease over time in the penetration rate along the centerline can be attributed to a gradual compaction of the aggregate base with traffic. Table 8 presents average penetration rates for the three sections. The data show that the penetration rates obtained for the aggregate subbase layer were 30 to 40 percent lower compared to those of the aggregate base. The subgrade showed the highest penetration rates, which were about 6 to 8 times those of the aggregate base and subbase.

Table 8 DCP Penetration Rates for Unbound Layers

Test Section	Area	Penetration Rates (mm/blow count)		
		Aggregate Base	Aggregate Subbase	Subgrade
543 Drained	Trafficked and Non-Trafficked	2.8	2.1	22.3
544 Undrained	Trafficked	2.0	1.1	13.3
	Non-trafficked	2.2	1.7	18.8
545 Undrained	Trafficked	2.0	1.7	15.8
	Non-trafficked	2.6	1.5	14.8

8.6.3 Open Test Pits

Open test pits in all the sections indicate that the aggregate base significantly contributed to the surface rutting in all the test sections.

The open test pit in Section 543 clearly showed the intrusion of fines from the aggregate base into the ATPB layer.

Density measurements using a nuclear density gauge indicated the relative compaction of the aggregate base to be in the range 98 to 101 percent. This is based on a maximum wet density of 2.42 g/cm^3 (151 lb/ft.^{3}) obtained for the aggregate base according to Caltrans Method 216.

9.0 COMPARISON OF PAVEMENT PERFORMANCE

Table 9 summarizes the number of ESALs to failure (as defined in Section 7.4.2 of this report) for surface rutting and cracking.

Table 9 Estimated ESALs (Millions) for Failure Criteria

Section	ESALs to Failure (× 10^6)	
	Rutting Failure	Cracking Failure
543 ARHM-GG/Drained	18.7^f	35.8
544 ARHM-GG/Undrained	20.3	14.3^f
545 DGAC/Undrained	37.3*	21.3^f

* extrapolated from data
^{f} section failed by this failure criterion

9.1 Drained versus Undrained Section Performance

As seen in Table 9, the three pavement sections had similar pavement lives when both the rutting and fatigue criteria are considered. These ranged from 14.3 × 10^6 to 21.3 × 10 ESALs when the smaller of the two ESAL values for each section is selected.

For this comparative calculation of pavement types, the failure mode was different for the undrained sections versus the drained section, with the amount of fatigue cracking governing performance in the undrained sections and rutting governing performance in the drained section. From the evidence presented, the stripping occurring in the ATPB under the heavy HVS loading accelerated the development of rutting in the drained section. Available evidence indicates that the stripped ATPB in the saturated condition was less resistant to rutting than the aggregate base in a wet (approaching saturation) condition.

9.2 ARHM versus DGAC Wearing Course Performance

Performance as measured by fatigue cracking of the DGAC wearing course was slightly better than that of the ARHM-GG. While design thickness of the DGAC wearing course was 75 mm, the actual thickness averaged 90 mm based on core measurements and thickness measurements of the exposed layers in the test pit. The additional thickness in the DGAC wearing course may have retarded crack propagation in the surface. Moreover, the ARHM-GG was found to have been poorly compacted during construction. Regardless of these differences, the tests indicate that the half thickness of the ARHM-GG as an overlay wearing course performed in a similar matter to the DGAC wearing course.

9.3 Dry versus Wet Condition Performance

The number of ESALs to failure as defined in this paper for similar pavement sections previously tested under dry condition is shown in Table 10.(*7-11*)

Table 10 Estimated ESALs (Millions) for Failure Criteria for Sections Under Dry Experiment

Section	ESALs to Failure ($\times 10^6$)		
	Rutting Failure	Cracking Failure	Total Applied ESALs
500 Drained	41.0^f	55.0	112.0
501 Undrained	109.0	34.0^f	59.0
502 Drained	100.0	91.0^f	117.0
503 Undrained	129.0	34.0^f	81.0

^{f} section failed by this failure criterion

The main structural difference between the sections tested under dry conditions and those tested under wet conditions is that the sections tested under dry conditions did not have the additional wearing course. With the exception of Section 500, because of the higher test temperatures that contributed to a rapid rutting failure, all other sections failed by fatigue cracking. Pavement life of the undrained sections under dry conditions was about 1.6 to 2.0 times the pavement life of the sections under wet conditions. This difference may have been even larger if the sections in the wet experiment were tested without the additional wearing course. A comparison of the results presented in Tables 9 and 10 also indicates that for the drained sections, wet conditions significantly reduced the structural benefit of the ATPB layer observed under dry conditions.

10.0 CONCLUSIONS

1. The asphalt treated permeable base layer placed between the asphalt concrete and the aggregate base layers used in this study stripped in the presence of water and heavy loading.
2. In the HVS testing, the asphalt treated permeable base layer clogged with fines from the underlying aggregate base thereby reducing the permeability of the ATPB layer

from 1 cm/s to 1×10^{-3} cm/s. This led to a saturation condition within the pavement that quickly contributed to the deterioration of the performance of the pavement section.

3. During the initial HVS loading cycles, the asphalt treated permeable base layer was effective in reducing elastic deflections at the pavement surface—the contribution to surface deflection attributed to the unbound layers was about 75 percent as compared to more than 90 percent for the section with the unbound layers. However, with the rapid deterioration of the ATPB layer under the heavy loading in the saturated condition, the drained section quickly reached similar levels of surface deflections as the undrained sections.
4. Permanent deformation was also reduced in the drained pavement sections until the onset of stripping in the ATPB.
5. The drained and undrained sections in the wet experiment failed by two different modes of failure: the drained sections failed by surface rutting and the undrained sections failed by fatigue cracking.
6. The performance of the two overlays (wearing courses) is similar regardless of the increased thickness in the DGAC and poor compaction of the ARHM-GG.

11.0 RECOMMENDATIONS

1. It is recognized that the ATPB can provide an increased structural capacity to asphalt concrete pavements in dry environmental conditions. However, long-term structural capacity requires that the ATPB be resistant to stripping, loss of cohesion, and stiffness reduction from water damage under saturated base conditions.

2. The performance of the ATPB can be substantially improved through improved mix design, drainage design, construction procedures, and maintenance practice. Increased asphalt contents and use of asphalt rubber binders may reduce the susceptibility of the ATPB to water damage.
3. It is also recommended that the use of the ATPB to intercept water entering through the pavement surface be reconsidered. As an alternative, if the permeability of the asphalt concrete is reduced by means of improved compaction, incorporation of sufficient asphalt concrete thickness to mitigate the potential for load associated cracking, and the degree of compaction increased in the aggregate base to a level sufficient to reduce the degree of rutting in these layers, then the requirement for placement of an ATPB layer between the AC and the AB would be eliminated.
4. If ATPB-type materials are to remain effective as a drainage layers, it will be necessary to ensure that adequate filter layers are provided adjacent to the ATPB to minimize the intrusion of fines and that edge and transverse drains are maintained to prevent becoming clogged with fines.

12.0 REFERENCES

1. California Department of Transportation. *Highway Design Manual, Fifth Edition.* State of California, Sacramento, CA, 1995.

2. Wells, G. *Evaluate Stripping of Asphalt Treated Permeable Base.* Minor Research Report No. 65332-638047-39303, F93RM01, California Department of Transportation, Sacramento, June 1993.

3. Hassan, H. and T. White. *Location the Drainage Layer for Bituminous Pavements in Indiana.* Draft Final Report No. FHWA A/IN/JHRP96, Purdue University Joint Highway Research Project for the Indiana Department of Transportation, West Lafayette, August 1996.

4. Hajek, J., T. Kazmierowski, H. Sturm, R. Bathurst, and G. Raymond. "Field Performance of Open-graded Drainage Layers." *Transportation Research Record.* National Research Council, Washington D.C., Vol. 1354, 1992.

5. Harvey, J., B. Tsai, F. Long, and D. Hung. *CAL/APT Program -- Asphalt Treated Permeable Base (ATPB): Laboratory Testing, Performance, Predictions, and Evaluation of the Experience of Caltrans and Other Agencies.* Report to California Department of Transportation, Pavement Research Center, Institute of Transportation Studies, University of California, Berkeley, July 1999.

6. Lovering, W. and H. Cedergren. "Structural Section Drainage." *Proceedings of the First International Conference on the Structural Design of Asphalt Pavements.* Ann Arbor, University of Michigan, 1962.

7. Harvey, J., L. du Plessis, F. Long, J. Deacon, I. Guada, D. Hung, and C. Scheffy. *CAL/APT Program: Test Results from Accelerated Pavement Test on Pavement Structure Containing Asphalt Treated Permeable Base (ATPB) - Section 500RF.* Report No. RTA-65W4845-3 prepared for the California Department of Transportation, Institute of Transportation Studies, University of California, Berkeley, January 1996.

8. Harvey, J., L. du Plessis, F. Long, J. Deacon, I. Guada, D. Hung, and C. Scheffy. *CAL/APT Program: Test Results from Accelerated Pavement Test on Pavement Structure Containing Untreated Base - Section 501RF.* Report No. RTA-65W4845-3 prepared for the California Department of Transportation, Institute of Transportation Studies, University of California, Berkeley, September 1997.

9. Harvey, J., L. du Plessis, F. Long, J. Deacon, I. Guada, D. Hung, and C. Scheffy. *CAL/APT Program: Test Results from Accelerated Pavement Test on Pavement Structure Containing Asphalt Treated Permeable Base (ATPB) - Section 502CT.* Report No. RTA-65W4845-3 prepared for the California Department of Transportation, Institute of Transportation Studies, University of California, Berkeley, January 1996.

10. Harvey, J., L. du Plessis, F. Long, J. Deacon, I. Guada, D. Hung, and C. Scheffy. *CAL/APT Program: Test Results from Accelerated Pavement Test on Pavement Structure Containing Untreated Base - Section 503RF.* Report No. RTA-65W4845-3 prepared for the California Department of Transportation, Institute of Transportation Studies, University of California, Berkeley, September 1997.

11. Harvey, J., L. Louw, I. Guada, D. Hung, and C. Scheffy. "Performance of CAL/APT Drained and Undrained Pavements under HVS Loading." *Transportation Research Board*, National Research Council, Washington D.C., Washington, 1998.

12. Harvey, J., N. Coetzee and L. Louw. *Design and Construction of CAL/APT Goal 3 DGAC and ARHM-GG Overlays, and Review of Caltrans Design and Construction Methods.* Draft Report for California Department of Transportation. Pavement Research Center, Institute of Transportation Studies, University of California, Berkeley, December 1999.

13. Bejarano, M. O., J. T. Harvey, A. Ali, M. Russo, D. Mahama, D. Hung, and P. Preedonant. *Performance of Drained and Undrained Flexible Pavement Structures under Wet Conditions – Accelerated Test Data from Test Section 543 (Drained).* Draft Report for California Department of Transportation. Pavement Research Center, Institute of Transportation Studies, University of California, Berkeley. Publication Pending.

14. Bejarano, M. O., J. T. Harvey, A. Ali, M. Russo, D. Mahama, D. Hung, and P. Preedonant. *Performance of Drained and Undrained Flexible Pavement Structures under Wet Conditions – Accelerated Test Data from Test Section 544 (Undrained).* Draft Report for California Department of Transportation. Pavement Research Center, Institute of Transportation Studies, University of California, Berkeley. Publication Pending.

15. Bejarano, M. O., J. T. Harvey, A. Ali, M. Russo, D. Mahama, D. Hung, and P. Preedonant. *Performance of Drained and Undrained Flexible Pavement Structures under Wet Conditions – Accelerated Test Data from Test Section 545 (Undrained).* Draft Report for California Department of Transportation. Pavement Research Center, Institute of Transportation Studies, University of California, Berkeley. Publication Pending.

16. Harvey, J. et al. *Initial CAL/APT Program: Site Information, Test Pavement Construction, Pavement Materials Characterizations, Initial CAL/HVS Test Results, and Performance Estimates.* Report No. RTA-65W485-3 prepared for the California Department of Transportation, Institute of Transportation Studies, Pavement Research Center, University of California, Berkeley, June 1996.

17. Russo, M. A. "Laboratory and Field Tests on Aggregate Base Material for Caltrans Accelerated Pavement Testing Goal 5." Master's thesis, University of California at Berkeley. 2000.

18. Harvey, J., M. O. Bejarano, A. Fantoni, A. Heath, and H. C. Shin. *Performance of Caltrans Asphalt Concrete and Asphalt-Rubber Hot Mix Overlays at Moderate Temperatures-Accelerated Pavement Testing Evaluation.* Draft report submitted to California Department of Transportation. Pavement Research Center, CAL/APT Program, Institute of Transportation Studies, University of California, Berkeley, July 2000.