APPLICATION OF CAL/APT RESULTS TO LONG LIFE FLEXIBLE PAVEMENT RECONSTRUCTION

Paper Submitted to Accelerated Pavement Testing Conference, Reno, NV, 1999

Paper No. GS3-2

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ABSTRACT
The California Department of Transportation (Caltrans) needs to rehabilitate or reconstruct much of its network of urban freeway pavements. A strategy has been developed by the Caltrans Accelerate Pavement Testing (CAL/APT) program to reconstruct these pavements with a flexible pavement incorporating design features that minimize the thickness of the pavement and improve constructability. The strategy meets Caltrans objectives for “Long-Life Rehabilitation Strategies” of 30 years service life and minimum maintenance.

This paper describes the strategy and discusses validation and calibration analyses based on accelerated pavement testing using the Heavy Vehicle Simulator, laboratory tests, and mechanistic-empirical analyses. Elements of the strategy evaluated include use of a “Rich Bottom” asphalt concrete layer with enhanced fatigue properties, improved compaction of the asphalt concrete, use of rut resistant mixes in the critical zone for mix rutting, and subgrade rutting criteria. An example design is presented in the paper.

INTRODUCTION AND BACKGROUND

The objectives of this paper are to:

- Present a strategy for reconstruction of heavy-duty pavement structures in urban areas of California,
- Discuss the basis for the elements of the proposed strategy, developed primarily through accelerated pavement tests, laboratory tests, and mechanistic-empirical analyses, and
- Present an example of implementation of the strategy in Los Angeles.

This strategy is intended to meet the California Department of Transportation (Caltrans) objectives for Long Life Pavement Rehabilitation Strategies.

Verification and calibration of the elements of the reconstruction strategy and the method for design of the asphalt concrete layer are based primarily on work performed as part of the Caltrans Accelerated Pavement Testing (CAL/APT) program. These elements include:

- Compaction of the asphalt concrete layer to 5 percent air-voids,
- A “Rich Bottom” layer, 50 to 75 mm thick at the bottom of the asphalt concrete,
- Exclusion of the Rich Bottom layer or any rut susceptible mixes from the critical zone for rutting within 100 to 150 mm of the surface,
- Design for rutting of the unbound pavement layers, and
- Use of thicker asphalt concrete layers instead of an asphalt-treated permeable layer (ATPB) just beneath the asphalt concrete.

Need for Long Life Pavement Reconstruction Strategies

The California Department of Transportation (Caltrans) operates a state highway network of more than 24,000 centerline kilometers, with over 78,000 lane-kilometers of pavements. In 1995, approximately 22,500 lane-kilometers, nearly 30 percent, required corrective maintenance or rehabilitation. Nearly 7,000 lane-kilometers required immediate attention to avoid rapid deterioration or loss of the facility. The California highway network is comprised of 68 percent flexible and 32 percent rigid in terms of total lane-kilometers. Of the total pavement lane-kilometers needing rehabilitation, 52 percent are flexible and 48 percent are rigid. Of the total pavement lane-kilometers required immediate attention, 49 percent are flexible and 51 percent are rigid. (*1*)

Although the majority of the Caltrans network consists of flexible pavements, most heavy-duty truck pavements in the urban areas are rigid. Approximately 90 percent of the rigid pavement lane-kilometers operated by Caltrans were constructed in the 15 years between 1959 and 1974, and were designed for 20 year lives based on traffic volumes and loads estimated at that time. (*2*) Approximately 80 percent of the rigid pavements needing rehabilitation are in urban areas in Southern California, with most of the remainder in urban areas in Northern California.

Most of the rigid pavements in need of reconstruction or rehabilitation have one of the structures shown in Figure 1. These structures consist of Portland Cement Concrete (PCC) slabs on cement treated base (CTB) and aggregate subbase (ASB).

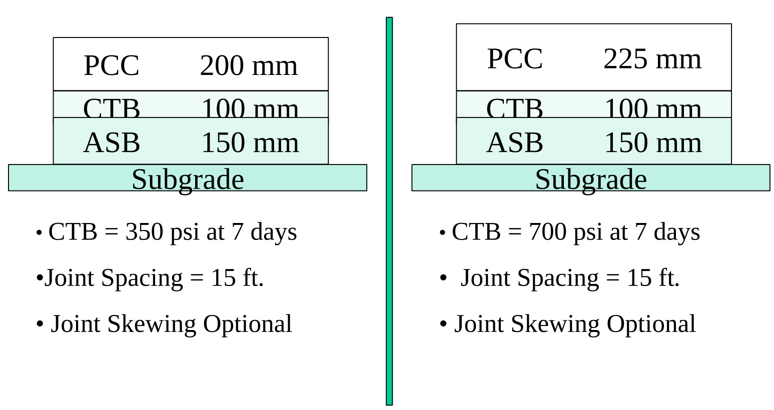


Figure 1. Typical rigid pavement structures from 1955 to 1964 (Caltrans design methods).

Currently, the most commonly used rehabilitation strategy for these pavements is to place a 107 mm asphalt concrete (AC) overlay on the cracked and seated slabs, with a fabric near the bottom of the overlay to delay reflection cracking. It is estimated that this may provide service lives of 6 to 15 years (*2*), with an assumed design life of 10 years.

Long life pavement rehabilitation strategies are defined by Caltrans as those that will provide 30 or more years of life with low maintenance needs. Some long-life strategies being considered by Caltrans and investigated by CAL/APT and industry include rigid pavement reconstruction, reconstruction of concrete pavements with asphalt concrete, and crack-seat and overlay strategies that provide longer life than the current practice.

All of the long-life strategies being evaluated must meet difficult requirements for both pavement life and constructability in heavily trafficked urban locations. Traffic criteria being used by Caltrans for identification of candidate projects for long-life strategies are either 150,000 average daily traffic or 15,000 average daily truck traffic. (*3*) Constructability is controlled by the ability to fit the new pavement structure into the existing infrastructure – which includes issues such as bridge heights, underground drainage and utilities, guard rails, and signage – and the extent to which traffic delays are caused by closing lanes to permit the reconstruction or rehabilitation work. High construction productivity is vital to minimizing traffic delay.

Proposed Strategy

The reconstruction strategy described in this paper involves removal of existing rigid pavement PCC slabs and CTB and replacement with a long-life flexible pavement (Figure 2).

If the required asphalt concrete thickness is less than the existing thickness of rigid pavement by more than 100 mm, then the existing aggregate subbase (ASB) may be recompacted to provide a working platform. This would typically occur if the required asphalt concrete thickness is less than 350 mm. The replacement asphalt concrete includes a bottom layer 50 to 75 mm thick, termed the "Rich Bottom," that has improved fatigue properties. Use of the Rich Bottom and improved compaction of the rest of the asphalt concrete permit thicknesses less than or equal to the thickness of the existing pavement structure. This strategy enhances constructability because it eliminates the need to recompact the subgrade and results in the same surface elevation as the existing rigid pavement. The compaction and Rich Bottom elements of the strategy also have some applications for reconstruction of flexible pavement structures and asphalt concrete overlays of flexible and rigid pavements.

DESIGN AND CONSTRUCTION REQUIREMENTS FOR LONG LIFE RECONSTRUCTION

To be implemented, a long life strategy should produce the desired design life and at the same time minimize disruption to the public.

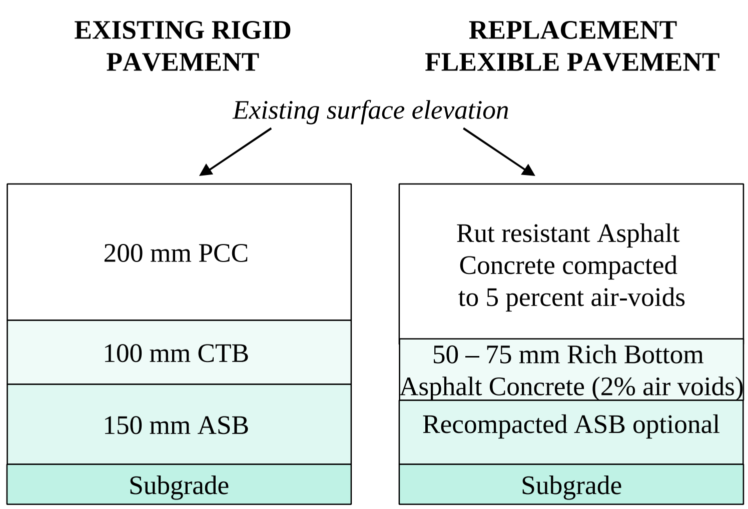


Figure 2. Proposed Reconstruction Strategy

Long-Life Design Requirements

The traffic associated with a design life of 30 years will vary depending upon the project location and growth in truck traffic volumes and axle loads. Truck traffic is expected to increase over the next 30 years due to high population growth and a strong trade based economy in the state. Table 1 shows traffic and truck traffic volumes along a few selected major urban freight corridors in 1995.

Table 1 Traffic and truck traffic on some of the Intermodal Corridors of Economic Significance (ICES) routes in urban areas of California. (*1*, *4*)

Route Number	County	1995 Average Daily Traffic (max)	1995 Daily Trucks in Design Lane (max)	30 Year ESALs current max truck traffic *	30 Year ESALs current max truck traffic* 20% increase	30 Year ESALs double current max traffic *
99	Sacramento	155,000	3,990	53,363,155	114,763,222	80,044,733
580	Alameda	164,000	4,189	56,024,626	120,487,002	84,036,939
10	LA	309,000	5,400	72,220,812	155,318,647	108,331,218
60	LA	287,000	9,975	133,407,888	286,908,056	200,111,832
710	LA	198,000	9,560	127,857,585	274,971,531	191,786,378
60	Riverside	147,000	4,161	55,650,148	119,681,646	83,475,222
10	San Bernardino	238,000	5,595	74,828,785	160,927,376	112,243,178
5	San Diego	166,000	1,568	20,970,784	45,099,933	31,456,176
405	Orange	327,000	4,742	63,420,572	136,392,782	95,130,858

* calculated with San Joaquin axle distribution

Assuming an axle load spectrum from Interstate 5 in San Joaquin county to be representative for many important corridors, 30-year design ESALs were calculated for the sections in Table 1 using the existing truck traffic volumes. Two additional cases were also evaluated as potential boundary conditions for thickness design (Table 1). To simulate the effect of potential increases in truck axle loads over the next 30 years, axle loads in the spectrum were increased by 20 percent. To simulate potential growth over the next 30 years, 30-year design ESALs were also calculated assuming linear growth in the number of trucks resulting in a doubling of traffic at the end of the 30-year period. From Table 1, it can be seen that a 30-year design life will likely require pavements to survive approximately 20,000,000 to more than 250,000,000 ESALs.

The long-life flexible pavement structures must be designed for fatigue cracking originating at the bottom of the asphalt concrete layer and rutting of the unbound layers. Good bonding between asphalt concrete lifts must be ensured to prevent critical tensile strains from occurring between lifts. This can be achieved by the use of tack coats between lifts.

It is assumed that thin renewable sacrificial surface wearing courses will be used to resist surface wear, provide skid and splash resistance, and retard the progression of cracks originating at the surface. It is likely that these surface courses will be a thin layer of open graded asphalt concrete, rubberized open graded asphalt concrete, or rubberized gap graded asphalt concrete. The surface courses will likely be removed and replaced approximately every 7 to 10 years during overnight traffic closures.

No-freeze environments characterize the locations of urban long-life projects, with rainfall between about 200 and 500 mm per year. (*5*) The seven-day average maximum pavement temperatures at 50 mm depth in the pavement are typically between 45 and 55°C. Some of the candidate projects are located inland from Los Angeles and in the Central Valley, where 7-day average maximum pavement temperatures at 50 mm depth are typically between 55 and 62°C. (*6*)

In the San Francisco Bay Area, most subgrades are either alluvial deposits of silty clay and gravel or relatively stiff clays. In the Los Angeles Basin and inland areas, most subgrades are sandy gravels or silty sands, except for some coastal areas that have clay subgrades. Much of the Central Valley has silty clay subgrades. Most of the long-life candidate locations do not have extremely soft, plastic subgrades.

Constructability Requirements

Construction productivity for the proposed strategy is improved by the following features:

- Pavement thicknesses that are less than the thickness of existing rigid pavement in order to eliminate recompaction of the subgrade and to avoid interference with subsurface drainage systems, conduits, and other subsurface infrastructure
- Keeping the reconstructed road surface elevation at or slightly above that of the existing rigid pavement in order to eliminate the need to raise bridges, drainage features, guard rails, ramps, etc.

In order to include these design features, the reconstructed pavement thicknesses must be somewhat less than the existing rigid pavement thicknesses of approximately 450 to 525 mm. A more detailed study to examine critical paths and construction production rates for several reconstruction projects in California urban areas for flexible and rigid long-life reconstruction strategies is currently underway. However, the features listed above must be included for any strategy in order to avoid reconstruction of much of the surrounding infrastructure.

ELEMENTS OF PROPOSED STRATEGY DEVELOPED OR VALIDATED BY CAL/APT

Accelerated Pavement Test Structures

Two pavements were constructed in April, 1995 at the Pavement Research Center located at the University of California at Berkeley Richmond Field Station. One pavement was a Caltrans “undrained” structure consisting of the prepared subgrade, an aggregate subbase and base layers, and an asphalt concrete surface. The second pavement was a Caltrans “drained” structure, in which a 75 mm thick layer of asphalt treated permeable base (ATPB) was placed beneath the asphalt concrete, replacing a portion of aggregate base.

The original drained and undrained pavements have each been used for two Heavy Vehicle Simulator (HVS) tests at a temperature of 20°C at the pavement surface. These tests were conducted with the goal of assessing the performance of the two structures with respect to fatigue of the pavement and rutting of the unbound layers at moderate temperatures. These tests are indicated in Figure 3 as Sections 500RF, 501RF, 502CT, and 503RF. The undrained pavement was also used for one test of mix rutting performance at a pavement surface temperature of 40°C (Section 504RF, Figure 3).

Both pavement structures were overlaid in April, 1997. (*7*) One side of both the drained and undrained pavements was overlaid with gap-graded asphalt rubber hot mix (ARHM). The other side of both structures was overlaid with asphalt concrete approximately twice as thick as the ARHM (Figure 3). Nine HVS tests were performed at elevated temperatures to evaluate rutting performance on the overlays (Sections 505RF through 513RF, Figure 3).

The overlays have been tested at a temperature of 20°C at the pavement surface to evaluate performance – primarily fatigue cracking and rutting of the unbound layers – at moderate temperatures (Sections 514RF, 515RF, 517RF and 518RF, Figure 3). First-level analysis has been completed on all of these sections.

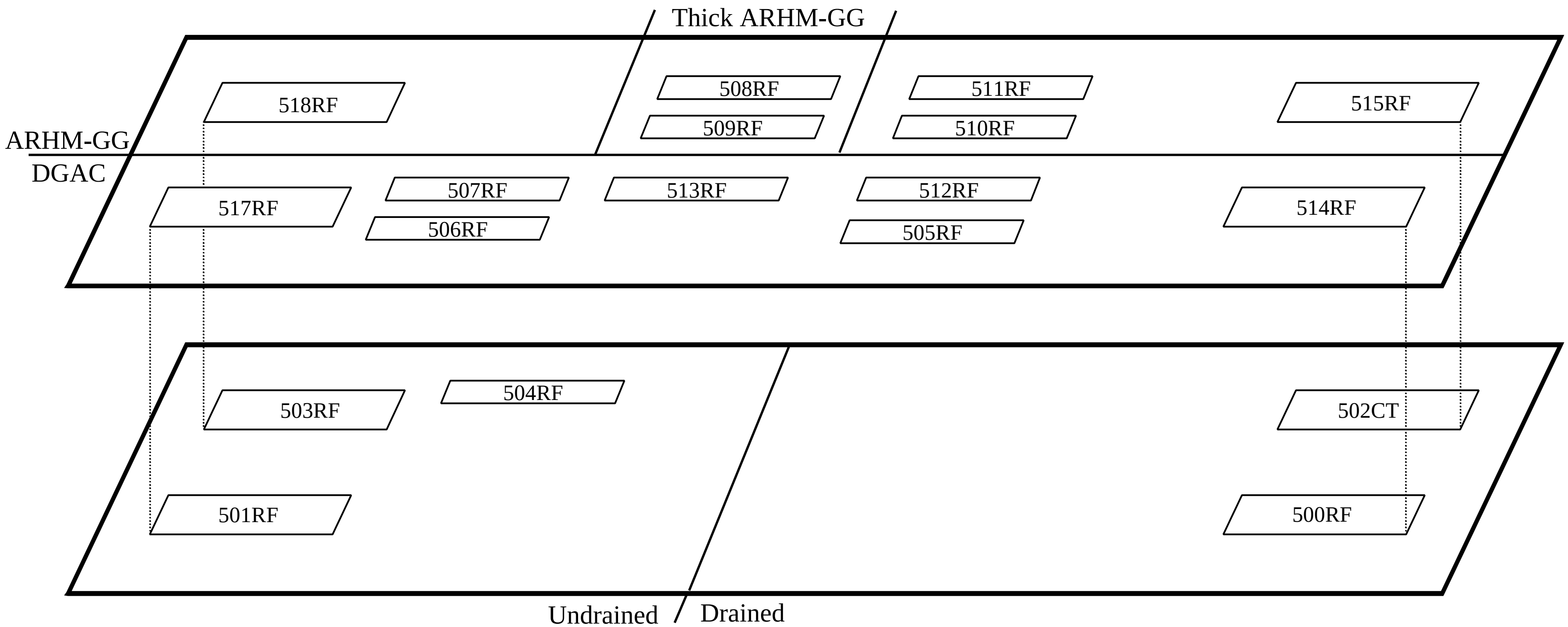
All of the tests have been conducted in essentially “dry” conditions, with no rainfall permitted to fall on the pavements. The purpose of the use of the dry conditions was to evaluate the structural benefits of the ATPB layer in conditions where little rainfall occurs. Caltrans requires use of the drained structure statewide regardless of environmental conditions. The functional performance of the drainage in the two structures and their structural performance under wet conditions will be tested beginning in the summer of 1999. Drainage systems are in place to intercept lateral flow to a depth of about 1 m below the surface of the pavements. However, the pavements are built on the native subgrade, and subject to capillary action from the groundwater table.

Improved AC Compaction

Laboratory Tests and Pavement Analyses

Caltrans construction specifications for asphalt concrete compaction have typically resulted in constructed air-void contents of 9 percent on average using end-result specifications, and 12 percent on average using the previous method specification. (*8*) Air-void contents have been reduced in recent years as Caltrans has moved from a method specification that prescribed compaction temperatures and passes of specified rollers, to an end-result specification. In the past two years, Caltrans has implemented a QC/QA system, which has passed responsibility for quality control from Caltrans to the contractor for most larger projects. (*9*) The recent experience with QC/QA and end-result specifications has demonstrated that better compaction can be obtained through improved control of the construction process.

As part of CAL/APT, a mix design and analysis system for fatigue of asphalt concrete pavements developed as part of the Strategic Highway Research Program was calibrated for California conditions, typical Caltrans flexible



500RF Fatigue test, dual bias, original AC, drained
501RF Fatigue test, dual bias, original AC, undrained
502CT Fatigue test, dual bias, original AC, drained
503RF Fatigue test, dual bias, original AC, undrained
504RF Rutting test, dual bias, AC, undrained
505RF Rutting test, dual bias, 61mm DGAC, drained
506RF Rutting test, dual radial, 76mm DGAC, undrained
507RF Rutting test, wide base single, 76mm DGAC, undrained
508RF Rutting test, wide base single, 61mm ARHM-GG, undrained
509RF Rutting test, dual radial, 61mm ARHM-GG, undrained
510RF Rutting test, dual radial, 37mm ARHM-GG, drained
511RF Rutting test, wide base single, 37mm ARHM-GG, drained
512RF Rutting test, wide base single, 76mm DGAC, drained
513RF Rutting test, aircraft tire, DGAC, undrained
514RF Fatigue test, dual radial, 62mm DGAC, drained
515RF Fatigue test, dual radial, 37mm ARHM-GG, drained
517RF Fatigue test, dual radial, 75mm DGAC, undrained
518RF Fatigue test, dual radial, 37mm ARHM-GG, undrained

Figure 3. Plan view of Pavement Research Center HVS test sections.

pavement structures, and a “standard” California dense graded asphalt concrete mix. (*10-12*) The “standard mix” includes California Valley AR-4000 asphalt, which has wide application in California. In the laboratory, relations were developed for asphalt concrete stiffness and beam fatigue life at 20°C for the standard mix. The mix design and analysis system is used to calculate the number of equivalent 80 kN single axle loads (ESALs) to fatigue cracking failure of the pavement.

Sensitivity analyses performed using the fatigue analysis system and the laboratory results for the standard mix indicate that increasing construction compaction has a very large effect on predicted pavement fatigue life. The improvement in pavement fatigue life calculated in the analyses for lower air-void contents is due to the combination of increased resistance to fatigue for a given tensile strain, and reduced tensile strains at the bottom of the asphalt concrete layer due to greater flexural stiffness. Pavements with thick asphalt concrete layers designed for 65 million to 85 million ESALs were found to obtain greater relative improvements from increases in compaction than pavements with thinner asphalt concrete layers. The thick asphalt concrete layers are the primary structural components controlling tensile strains in these pavements, and the tensile strains are particularly sensitive to the asphalt concrete stiffness.

The results of the laboratory testing and pavement fatigue life calculations provide clear indications of the effects of compaction on performance, however, further verification and calibration of the results was needed from testing closer in scale to field trafficking of in-service pavements. For example, a change of asphalt concrete air-void content from 8 to 5 percent showed a 200 percent increase in fatigue life for a pavement with a 320-mm thick asphalt concrete layer.

Based on these results, changes were recommended to Caltrans to reduce the allowable air-voids content as much as possible, while still maintaining an achievable specification. Questions that remained from the laboratory tests and pavement analyses were:

- Is it possible to compact typical Caltrans mixes to 5 percent or 2 percent air-voids (Rich Bottom layer)?
- Are increases in fatigue life of the magnitude indicated by the laboratory testing and analysis possible due to changes in compaction alone for a relatively thick asphalt concrete pavement?

HVS Tests and Analyses

The asphalt concrete layer of the drained and undrained pavements tested for Sections 500RF, 501RF, 502CT, and 503RF was built in two lifts. The design and the as-built layer thicknesses for all layers determined by coring and trenching performed to date are shown in Table 2. According to the Caltrans design procedure, the design life of these pavement structures is 1,000,000 ESALs. Important differences in the layer thicknesses can be seen for the asphalt concrete and subbase layers. The difference between design and as-built thicknesses for the subbase are due to variance in the elevation of the subgrade surface, and inclusion of a 0.5 percent longitudinal slope and 2 percent cross slope in the surface of the subbase. The subgrade is a clay material, with plasticity index ranging between 25 to 40 and liquid limit ranging between 40 and 55. The AASHTO classification is A-7-6, and the Unified System classification is CH.

Table 2 Design and actual pavement layer thicknesses (mm).

	Section 500RF		Section 501RF		Section 502CT		Section 503RF	
Layer	Design	As Built	Design	As Built	Design	As Built	Design	As Built
AC top	61	74	61	63	61	68	61	74
AC bottom	76	76	76	84	76	80	76	88
ATPB	76	76	none	none	76	76	none	none
AB	182	182	274	274	182	182	274	274
ASB	229	137	229	215	229	215	229	305

A commercial highway contractor, awarded the job based on low bid, constructed the pavements. The Caltrans method specification was used for compaction. Because of the short haul distance from the plant to the site, the short length of the test pavements, and good temperature control of the mix at the site, the asphalt concrete was compacted to lower air-void contents than are typically obtained in California. The air-void contents of the top asphalt concrete lift of the test pavements observed in a limited number of cores taken just outside the test sections indicate better compaction than is typically obtained in the field (Table 3). The air-void contents of the bottom lift of all sections, and of the top lifts in Sections 502CT and 503RF are even lower, approaching those that might be considered to potentially result in unstable mixes.

Table 3 Average construction air-void contents for HVS test sections.

Test Section	Top Asphalt Concrete Lift Air-Voids (percent)	Bottom Asphalt Concrete Lift Air-Voids (percent)
500RF	7.8	4.4
501RF	7.2	5.6
502CT	4.1	2.4
503RF	4.8	4.4

All four sections were subjected to a similar loading program: 40 kN dual-wheel load for 150,000 repetitions; 80 kN for 50,000 repetitions; and finally, 100 kN repetitions until failure. The terminal failure condition adopted for the sections was based on Caltrans Pavement Management System criteria. According to Caltrans pavement raters, when the testing was completed each section resembled a new pavement that had failed by alligator cracking and was in need of structural overlay. Therefore, fatigue cracking was the controlling failure criterion, however, surface deflection, surface rutting, and permanent deformation of the various layers were also evaluated.

Assuming the Caltrans load equivalence factor exponent of 4.2, the ESALs to 2.0-m/m^2 crack density, and the crack density at the end of trafficking are summarized for the four sections in Table 4. It can be seen that the test sections had far better performance than the 1,000,000 ESALs that the Caltrans thickness design procedure assumes.

Table 4 Summary of cracking performance for Sections 500RF, 501RF, 502CT and 503RF.

Test Section	Load Repetitions	Final ESALs	Final Crack Density (m/m^2)	ESALs to Crack Density of 2.0 m/m^2
500RF	2,572,372	112,400,000	2.5	55,000,000
501RF	1,426,467	58,600,000	9.6	34,000,000
502CT	2,673,589	117,100,000	4.0	91,000,000
503RF	1,911,823	81,400,000	6.5	34,000,000

A few cores drilled out of the sections after completion of the testing indicated that cracking was restricted to the upper lift and that the lower lift was sound. The coring also showed weak bonding between the two AC lifts, which helps explain why the upper was cracked instead of the lower lift.

Laboratory testing and pavement analyses were used to evaluate the differences between the design life assumed in the Caltrans thickness design and the observed performance of the HVS test sections. The multi-layered linear-elastic program CIRCLY (*13*) was used to model the pavement as a layered elastic system and to calculate critical tensile strains under the simulated traffic load. The load consisted of a 40-kN dual-wheel load with tires located 305 mm apart (center-to-center) and a contact stress of 690 kPa. The pavement layers were modeled according to their assumed characteristics for design of field sections – which would be expected to be saturated in many cases and therefore have lower stiffnesses – and their characteristics as built in the test sections.

The modulus values for the unbound materials, given in Table 5, were selected based on extensive laboratory and in-situ material testing reported previously. (*14*) The assumed modulus values given under “design conditions” are 80 percent of the measured values (“actual conditions”). In the absence of typical field modulus data for Caltrans materials, the 20 percent reduction was somewhat arbitrarily estimated based on judgment and evaluation of laboratory results of as-compacted, soaked, and saturated laboratory test results. *(14, 15)* The reduction is intended to reflect the different moisture conditions expected in the field as compared to the test section conditions inside the Pavement Research Center facility at the RFS. The subgrade layer was modeled as half space (semi-infinite thickness). The assumed Poisson’s ratio of the asphalt concrete was 0.35.

Table 5 Elastic moduli (MPa) of the supporting layers

Layer	Design Stiffness	As-built Stiffness	Assumed Poisson’s Ratio
ATPB	827	1034	.35
AB	240	300	.40
ASB	120	150	.35
SG	56	70	.35

The asphalt concrete mix used for four test sections, referred to as the “HVS mix,” was a Caltrans 19-mm maximum aggregate size mix with gradation similar to that of the “standard mix” used in the fatigue study. The asphalt is an AR-4000, probably from California Valley sources, though not the same refinery as the Valley asphalt "standard mix" binder. The asphalt content is 4.8 percent and constructed air-void content varies considerably for the various sections, as indicated in Table 3. Stiffness and fatigue life relations at 20°C were developed through laboratory testing of flexural beams compacted to different air-void contents from mix collected during construction of the HVS test sections.

A second "standard" mix was also used in the analysis. This mix is exactly the same as the "standard" mix described previously, except that the AR-4000 binder was from a California Coastal crude source. Stiffness and fatigue relations were developed through laboratory testing of beams compacted to different air-void contents and asphalt contents. (*15*)

Stiffnesses were calculated for the three mixes for a typical air-void content of 8 percent and the air-void contents encountered in the two lifts of asphalt concrete in the test sections (Table 6). Asphalt contents for the two standard mixes were assumed to be the optimum determined by the Hveem procedure.

Table 6 Stiffness moduli and actual air void content of various asphalt concrete mixes

		Stiffness at 20°C in MPa and air-void content (%) given in brackets				
Mix	AC lift	Design	Section 500	Section 501	Section 502	Section 503
Valley (AC=5%)	Upper	6,736 (8.0)	6,839 (7.8)	7,157 (7.2)	9,052 (4.1)	8,584 (4.8)
	Lower	6,736 (8.0)	8,848 (4.4)	8,079 (5.6)	10,296 (2.4)	8,848 (4.4)
Coastal (AC = 5 %)	Upper	1,899 (8.0)	1,946 (7.8)	2,094 (7.2)	3,059 (4.1)	2,808 (4.8)
	Lower	1,899 (8.0)	2,949 (4.4)	2,546 (5.6)	3,765 (2.4)	2,949 (4.4)
HVS (AC = 4.8%)	Upper	7,194 (8.0)	7,317 (7.8)	7,700 (7.2)	10,018 (4.1)	9,440 (4.8)
	Lower	7,194 (8.0)	9,766 (4.4)	8,820 (5.6)	11,574 (2.4)	9,766 (4.4)

The simulations indicate that the different performance of the HVS test sections as compared to the design performance assumption can be explained by differences between the design and as-built structures, in particular the superior compaction obtained on the HVS test sections compared to typical compaction obtained in the field (Table 7).

Table 7 Comparison of expected fatigue lives (millions of ESALs) for assumed design conditions and Valley standard mix versus as-built conditions and HVS mix.

Section type:	Drained sections (with ATPB)			Undrained sections (without ATPB)		
Modeled case:	Simulated		90 % design reliability	Simulated		90 % design reliability
Section name:	500RF	502CT		501RF	503RF	
Design Conditions — Valley Mix, 8 percent air-voids:	8.17	8.17	2.19	8.17	8.17	0.650
As-Built — HVS Mix, Full Bond, actual air-voids	360	787		80	262	
As-Built — HVS Mix, Frictionless, actual air-voids	8.02	43.3		7.9	20.7	

Note: Results differ slightly from those in References *16*, *17*, *18*, and *19* because of location of critical strain in calculations.

The HVS mix and as-built conditions were evaluated for a condition of full bonding between the two asphalt concrete lifts and for a condition of no friction between the two asphalt concrete lifts, with the actual condition lying somewhere between the two. Assuming that the Caltrans design estimate includes a reliability level of about 90 percent, it can be seen that the Caltrans design estimate for the two pavement structures of 1,000,000 ESALs is similar to that for the design conditions for the Valley standard mix. The drained structures have somewhat greater life due to the greater stiffness of the ATPB layer compared to that of the aggregate base it replaces.

The performance of the test sections under HVS loading can be seen to fall within the large range of predicted fatigue performance between full bond and frictionless analyses for the as-built conditions (Table 7). In the analyses, the frictionless bond case results in the critical tensile strain occurring at the bottom of the top lift of asphalt concrete and propagating upward, which coincides with the observation that cracking only occurred in the top lift. The actual bonding conditions were not frictionless, evidenced by observed aggregate friction at the interface between the lifts, although many areas of the test sections showed no cohesion.

The extent to which improved asphalt concrete compaction was responsible for the difference in performance between the HVS tests and the Caltrans design estimate can be assessed by separating the effects of compaction from those of layer thickness and the stiffness of the unbound layers in the analyses. It can be seen in Table 8 that changing from design conditions to as-built conditions for layer thickness and stiffness of the unbound layers results in increases in the simulated fatigue life for both the Valley and Coastal standard mixes, ranging between 230 and 600 percent.

Table 8 Comparison of expected fatigue lives (millions of ESALs) for assumed design conditions versus as-built conditions, and for assumed design asphalt concrete air-void contents and as-built air-void contents.

Section Type:	Drained Pavement Structure		Undrained Pavement Structure	
Section Name:	Section 500RF	Section 502CT	Section 501RF	Section 503RF
Design Conditions — Valley Mix, 8 percent air-voids:	8.17	8.17	2.62	2.62
Design Conditions — Coastal Mix, 8 percent air-voids:	126	126	5.05	5.05
As-built Conditions — Valley Mix, 8 percent air-voids:	23	23.4	6.14	10.6
As-built Conditions — Coastal Mix, 8 percent air-voids:	614	647	16.8	30.2
As-built Conditions — HVS Mix, 8 percent air-voids:	138	140	35	64
As-built Conditions — Valley Mix, as-built air-voids:	64	143	14	44.1
As-built Conditions — Coastal Mix, as-built air-voids:	1,105	2,224	43.6	171
As-built Conditions — HVS Mix, as-built air-voids:	360	787	80	262

Note: Results differ slightly from those in References *16*, *17*, *18*, and *19* because of location of critical strain in calculations.

Changing the air-voids content from the design value of 8 percent to the as-built air-void contents results in an increase in the simulated fatigue life for all three mixes. There is a synergistic effect between increased compaction and increased asphalt concrete thickness that results in increases of fatigue performance of 180 to 610 percent. It can be seen that there is also a tremendous difference in the expected performance of the Coastal asphalt mix compared to the Valley asphalt mix – this difference bears further investigation.

Rich Bottom Designs

Pavement Analyses

The beneficial effect of increased compaction on fatigue performance is evident, however, compaction of conventional asphalt concrete layers near the surface of the pavement may result in rutting of the mix in the field under conditions of heavy loads, high tire pressures, and hot climates. Resistance to the shear stresses, which are responsible for severe ruts, increases with compaction to a critical air-void content of about three percent for most dense graded mixes, and falls off rapidly for air-void contents below the critical value. (*20*)

Placement of fatigue resistant mixes near the bottom of the asphalt concrete layers of the pavement and use of more rut resistant mix near the surface has been standard practice in Australia for more than 15 years. A variation on this type of structure, termed "Rich Bottom," was evaluated for fatigue performance using the laboratory results and structures described previously. (*12*, *11*) In order to simplify construction, the Rich Bottom structure is assumed to use the same asphalt and aggregate used in the upper layers of asphalt concrete. The Rich Bottom layer asphalt content is increased by 0.5 percent over the target determined for the upper layers to prevent rutting of the mix and in order to facilitate compaction (Figure 2). The 50- to 75-mm thick Rich Bottom layer is compacted to an air-void content of 2 to 3 percent.

The Rich Bottom layer must be kept at least 100 mm below the surface of the pavement, where shear stresses are smaller and the probability of hot temperatures is reduced. This restriction limits the use of Rich Bottom structures to asphalt concrete layers of 150 mm and thicker. For use in reconstruction of existing rigid structures such as that shown in Figure 1, the need for asphalt concrete layers greater than 150 mm thick is a foregone conclusion.

Application of the Rich Bottom strategy was investigated using the fatigue analysis system and the pavement structures and standard mix relations for stiffness and fatigue life described previously. (*12*) The results indicated tremendous improvements in fatigue performance of structures with a Rich Bottom layer compared to conventional structures with the same total asphalt concrete thickness. These results are dependent on full bonding between all asphalt concrete layers, which forces the critical strains to occur at the bottom of the Rich Bottom.

HVS Tests and Analyses

After completion of HVS testing of the initial four test sections (500RF, 501RF, 502CT and 503RF) described above, the same were tested again after being overlaid. The thicknesses of the overlays and the loading on the overlays are shown in Table 9.

Table 9 Design overlay thicknesses for HVS test sections

Original HVS Test Number	Loading on Original Section	Design Overlay	HVS Test Number for Overlaid Section	Loading on Overlaid Section
500RF	150,000 reps at 40 kN, 50,000 reps at 80 kN, 2,372,732 reps at 100 kN	13 mm level up course*; 62 mm overlay of AC	514RF	171,517 reps at 40 kN, 317,141 reps at 80 kN, 1,348,898 reps at 100 kN
501RF	150,000 reps at 40 kN, 50,000 reps at 80 kN, 1,226,467 reps at 100 kN	75 mm overlay AC	517RF	148,737 reps at 40 kN, 178,075 reps at 80 kN, 2,019,026 reps at 100 kN
502CT	150,000 reps at 40 kN, 50,000 reps at 80 kN, 2,473,589 reps at 100 kN	37 mm overlay of ARHM	515RF	128,774 reps at 40 kN, 217,559 reps at 80 kN, 2,064,563 reps at 100 kN
503RF	150,000 reps at 40 kN, 50,000 reps at 80 kN, 1,711,823 reps at 100 kN	37 mm overlay of ARHM	518RF	Testing currently underway

* Leveling course of 9.5 mm maximum aggregate size [MSA] asphalt concrete, asphalt concrete overlay had 19 mm MSA, ARHM overlay had 12.5 mm MSA (Air-void contents and original HVS test descriptions in References *16*, *17*, *18*, and *19*).

Assuming the Caltrans load equivalency exponent of 4.2, the bottom lift of asphalt concrete in Section 502CT has been subjected to 117million ESALs. It has been subjected to and additional 101 million ESALs in Test 515RF, with the middle of the three lifts cracked. Extensive sampling and analysis of cracks indicates that the bottom lift of the asphalt concrete has generally not cracked in Section 502CT/515RF despite application of approximately 218 million ESALs. (*21*) In some locations, the bottom lift has begun to crack, but the cracks were observed to be reflecting downward halfway through the lift from the cracked asphalt concrete above. Two cores taken from Sections 500RF/514RF and 501RF/517RF show similar cracking patterns. In all of these sections, the overlay has been found to have cracked due to reflection of cracks from the top lift of the original asphalt concrete now in the middle of the total asphalt concrete layer.

Sections 500RF/514RF and 501RF/517RF have been subjected to a total of 181 million and 157 million ESALs, respectively, with no apparent cracking of the asphalt concrete layer from the bottom. The initial air-void contents in the bottom lift of the asphalt concrete layer are similar to those assumed for the Rich Bottom strategy, particularly for Sections 502CT and 503RF (Table 2). These results provide a degree of verification of the type of fatigue performance for the Rich Bottom strategy indicated by the laboratory tests and analyses. If good bonding had been achieved between the top and bottom lifts of the original asphalt concrete, calculations indicate that strains at the bottom of the asphalt concrete would be smaller.

Critical Zone for Surface Rutting

A partial verification that rutting of the mix primarily occurs in the upper 100 mm in pavements with the recommended levels of compaction in the asphalt concrete and in the Rich Bottom layer was obtained from the rutting tests performed at elevated temperatures on the original asphalt concrete and overlays. The conditions for the rutting tests are summarized in Table 10. (*7*)

Table 10 Rutting test sections included in evaluation of critical zone for rutting of the mix, and approximate conditions of tests.

Test Section:	504RF	505RF	506RF	507RF	508RF	509RF	510RF	511RF	512RF
Wheel Type:	Dual Bias		Dual Radial	Wide Single		Dual Radial		Wide Single	
Surface Mix: *	Orig. AC	AC	AC	AC	AR	AR	AR	AR	AC
Overlay Thickness (mm): **	none	60	78	74	67	70	39	40	54
Nominal Surface Temp. (°C):	45 ***	55	55	55	55	55	55	55	45

Notes: * all mix types refer to the overlays except for Section 504RF (AC = asphalt concrete, AR = asphalt rubber hot mix); ** from cores; *** nominal temperature.

Temperature was controlled during the tests based on thermocouples installed in the pavement. Average temperature profiles for several tests are summarized in Figure 4. It can be seen in Figure 4 that the temperature profiles were similar for the different tests. The wheel speed during the rutting tests averaged 7.6 kilometers per hour. The slow wheel speed and hot pavement surface temperatures represent severe conditions for rutting development, and are typical of congested traffic in the Los Angeles basin.

The results of the rut tests are shown in Figure 5. The majority of the vertical rut depth in each test occurred in the asphalt concrete and ARHM overlays, as can be seen in preliminary results from slabs removed from the tests sections as shown in Table 11.

Table 11 Preliminary measurements of rut depth in overlay, top asphalt concrete and bottom asphalt concrete layers (mm).

Section	Left Wheelpath *			Right Wheelpath *			Single Wheelpath *		
	Overlay	Top	Bottom	Overlay	Top	Bottom	Overlay	Top	Bottom
505RF	7	4	-	2	7	-	-	-	-
506RF	8	4	2	8	2	3	-	-	-
507RF	-	-	-	-	-	-	11	5	1
508RF	-	-	-	-	-	-	5	8	-
512RF	-	-	-	-	-	-	3	3	-

*Note: Left and right wheelpath applicable to sections tested with dual wheel; single wheelpath tested by wide-base single wheel

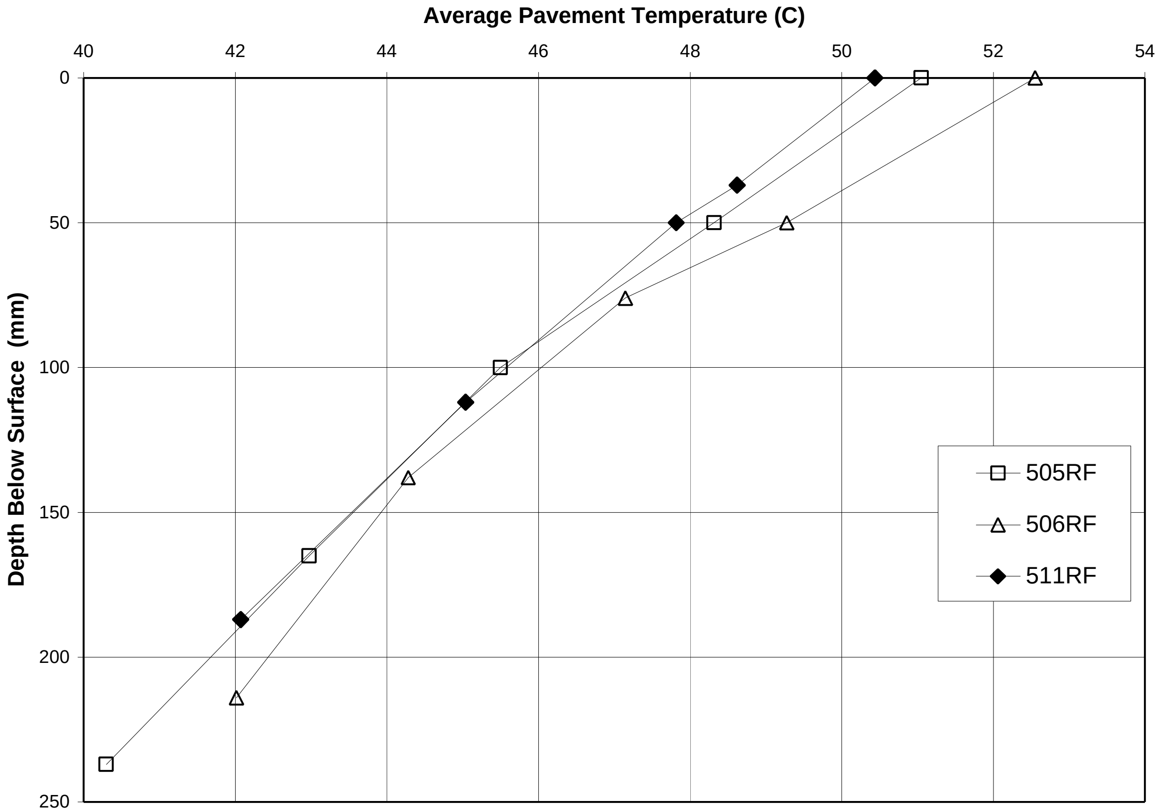


Figure 4. Average temperature profiles for HVS rutting tests.

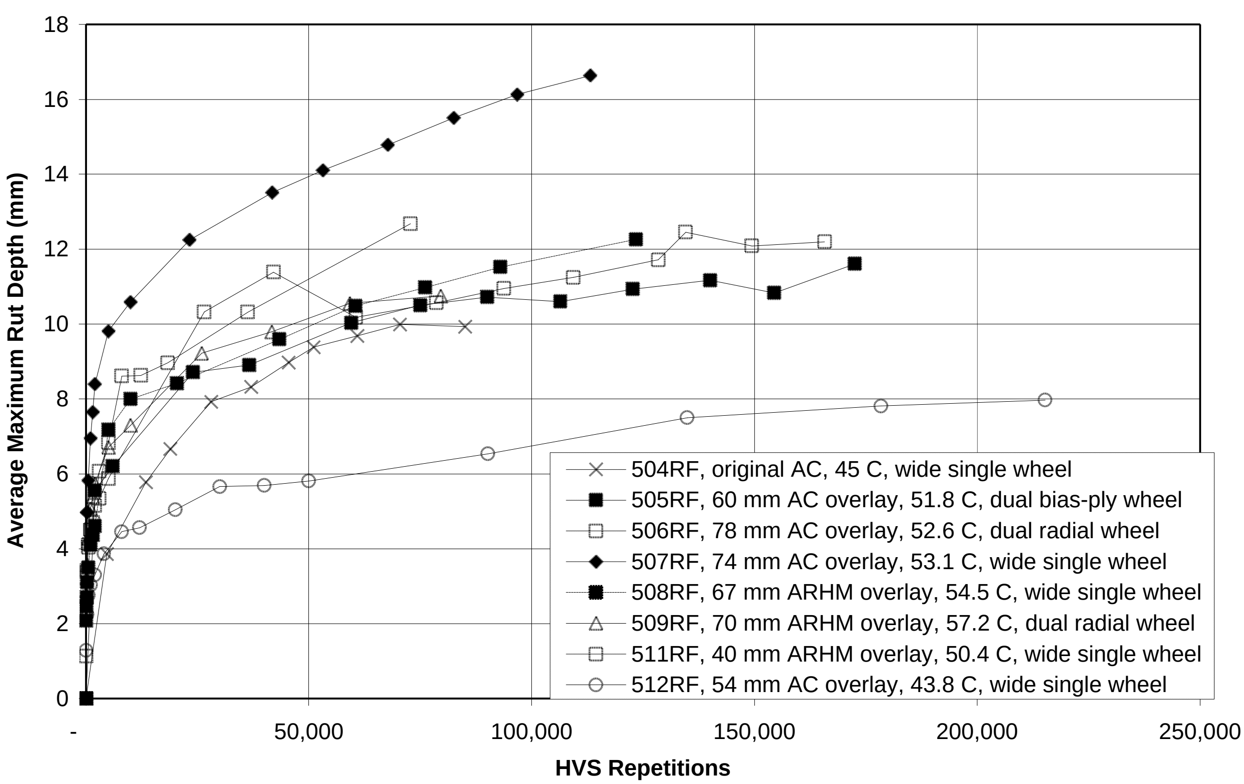


Figure 5. Average maximum rut depth versus HVS 40 kN wheel load repetitions

Cores were taken across the rutted test sections after trafficking, including the untrafficked, dilated “humps” at the edges of the ruts, and in the wheelpath. These cores were split into the three layers of overlay, top lift of original asphalt concrete and bottom lift of original asphalt concrete, and the air-void contents of each layer were measured. Preliminary air-void measurement results are shown in Table 12.

Table 12 Preliminary measurements of air-void content after trafficking from cores for asphalt concrete and asphalt rubber hot mix layers

Test	Overlay, Test Temperature, Wheel	Time of Coring	Top Original AC Lift Air Voids	Overlay Air Voids
505RF	60 mm ARHM, 51.8 C, dual	Before trafficking	4.7	4.0
		After trafficking	4.1	2.8
506RF	78 mm AC, 52.6 C, dual	Before trafficking	3.8	4.0
		After trafficking	3.5	2.6
507RF	74 mm AC, 53.1 C, single	Before trafficking	4.1	5.4
		After trafficking	2.7	2.3
508RF	67 mm ARHM, 54.5 C, single	Before trafficking	6.4	8.3
		After trafficking	6.7	6.0
509RF	70 mm ARHM, 55.2, dual	Before trafficking	8.7	7.2
		After trafficking	5.5	7.0
512RF	54 mm AC, 43.8 C, single	Before trafficking	6.7	8.5
		After trafficking	5.6	4.0
All Tests Average, Bottom Lift		Before trafficking	3.9	
		After trafficking	3.6	

The overlays and top lift of asphalt concrete, making up approximately the upper 115 to 150 mm of the pavement surface, experienced some densification. None of these layers had densified to an air-void content below 2.3 percent at the point at which a 12.5-mm rut had been reached or at the point at which the rutting had reached an asymptotic value. Densification was greatest in Section 512RF, which had the largest air-void content before trafficking despite being trafficked at a surface temperature that was 7 to 11°C less than that of the other sections. On the other hand, the bottom lift of the original asphalt concrete had very little densification (Table 12). Figure 4 indicates that pavement temperatures in the bottom lift were considerably cooler than in the overlays and top lift. At these depths, shear stresses responsible for shear deformation and that contribute to densification are also greatly reduced. (*22*)

The HVS test results indicate that the combination of reduced shear stresses, the well compacted mix, and cooler temperatures resulted in little densification or shearing of the nearly “Rich Bottom” layers at depths of 115 to 150 mm below the surface of the pavements. The effect of including an asphalt content 0.5 percent (by mass of aggregate) greater than the design asphalt content for rut resistance at the surface remains to be evaluated. However, for reconstruction of the Caltrans rigid pavement structures using a Rich Bottom pavement, the Rich Bottom layer would be expected to be placed below at least 200 mm of rut resistant asphalt concrete – at least 100 mm deeper than in the structures tested by the HVS to date.

Rutting of the Unbound Layers

Rutting can also occur in the subgrade and any other unbound layers below the asphalt bound layers. The potential for undesirable levels of rutting in the unbound layers is typically evaluated in terms of a limiting vertical strain at the top of the subgrade, a ratio between vertical stress and shear strength at a critical location in each unbound layer, or the elastic energy in the various unbound layers. (*23*, *24*) As can be seen in the example that follows in this paper, flexible pavement reconstruction of projects such as those included in Table 1 requires asphalt concrete layers typically thicker than 250 mm. The applicability of the existing criteria for rutting of the unbound layers to these pavements requires a large extrapolation because the criteria are primarily based on the AASHO Road Test and other test roads with less than 10,000,000 ESALs trafficking and asphalt concrete layers of less than 150 mm. (*23*)

Pavement Analyses

Calculations were performed to estimate the number of ESALs permitted by the Asphalt Institute subgrade strain criterion for Sections 500RF, 501RF, 502CT and 503RF (Table 13). (*23*) The calculations were performed using the same procedures, conditions, and pavement models shown for Table 7. The results are shown in Table 13.

Table 13 Comparison of expected allowable ESALs (millions) for Asphalt Institute subgrade strain criterion for assumed design conditions versus as-built conditions and air-void contents

Section Type:	Drained Pavement (with ATPB)		Undrained Pavement (without ATPB)	
Section Name:	500RF	502CT	501RF	503RF
Design Conditions — Valley Mix, 8 percent air-voids:	18.7	18.7	14.7	14.7
Design Conditions — Coastal Mix, 8 percent air-voids:	6.23	6.23	4.55	4.55
As-built Conditions — Valley Mix, as-built air-voids:	30.2	71.1	42.0	146
As-built Conditions — Coastal Mix, as-built air-voids:	8.46	24.1	12.6	46.0
As-built Conditions — HVS Mix, as-built air-voids:	33.6	81.0	46.0	164.0

Note: results differ slightly from those in References *16*, *17*, *18*, and *19* because of location of critical strain in calculations

HVS Tests and Analyses

The Multi-Depth Deflectometer (MDD) technology developed by CSIR and used by CAL/APT enables elastic and plastic response of the various layers to be monitored. By locating MDD modules at every layer interface, it is possible to measure the plastic deformation of the individual layers, and hence assess the contribution from each layer to the total surface deformation (rutting). These results are given in Table 14 as percentages of the total surface rut depth for the original four HVS tests on the drained and undrained pavements at 20°C.

The authors of the Asphalt Institute methodology state that when using the Asphalt Institute subgrade strain criterion, *"If good compaction of the pavement components is obtained and the asphalt mix is well designed, rutting should not exceed about 12.5 mm at the surface for the design traffic, N."* (*23*) The conservatism implied by this statement is borne out by comparison of the calculations shown in Table 13, with the results of the HVS tests shown in Table 14.

Table 14 Contribution of the various layers to the total surface rut depth (percent) for original four HVS tests on drained and undrained pavements assuming Caltrans load equivalent exponent of 4.2

Section:	500RF	501RF	502CT	503RF
Asphalt Concrete:	52 %	52 %	68 %	48 %
ATPB:	7 %	none	*	none
Aggregate Base:	17 %	26 %	16 %	33 %
Aggregate Subbase:	12 %	11 %	6 %	17 %
Subgrade:	12 %	11 %	10 %	2 %
80 ESALs (millions) at Finish: *	112	60	117	81
Rut Depth at Finish (mm):	15	11	14	11

* included with asphalt concrete layer in calculations

The results in Table 14 show that 48 to 68 percent of the observed surface rutting is due to the deformation of the asphaltic layers and that the contribution of subgrade plastic deformation to the total rutting is minimal. The aggregate base and subbase layers contributed 22 to 50 percent of the total vertical permanent deformation.

Although the conditions for these tests were idealized to some degree because of the condition of a constant temperature of 20°C, they do suggest that the subgrade strain criterion may be overly conservative for pavement sections with asphalt concrete layers thicker than 150 mm, as with HVS test sections 500RF, 501RF, 502CT, and 503RF. Development of less conservative design criteria for rutting of the unbound layers is needed.

Substitution of Thicker AC for ATPB Layer

The calculations of pavement fatigue lives included in Table 8 indicate that substitution of asphalt treated permeable base (ATPB) for aggregate base (AB) at a ratio of 1 to 1.27 results in greater fatigue life. To some degree these calculations are supported by the fatigue results for the first four pavement HVS test sections.

It will be difficult to install an ATPB layer below the asphalt concrete when reconstructing the existing rigid pavement sections because of the shallow depth available and the thick asphalt concrete layers required to provide adequate fatigue and rutting lives.

While adequate drainage for subsurface entry of water into the pavement system should be provided where necessary, water should be prevented from entering the pavement from the surface by limiting cracking through design of adequate asphalt concrete thicknesses and mixes. In addition, use of the Rich Bottom concept and the thickness and increased compaction of the rest of the asphalt concrete should greatly limit pumping of subgrade soils to the surface.

APPLICATION OF PROPOSED TECHNOLOGY TO A LONG-LIFE RECONSTRUCTION PROJECT

Pavement thicknesses for the proposed strategy for Interstate Route 710 in Los Angeles County were calculated using the University of California, Berkeley Fatigue Design and Analysis Procedure, assuming the hypothetical use of the two standard mixes described under "Improved AC Compaction." The pavements were designed for 200×10^6 ESALs – these thicknesses will be further refined using the actual mix recommended for use. The existing rigid pavement structure on Route 710, constructed in the late 1950s, is shown in Figure 1.

A range of resilient moduli for the subgrade was selected from Caltrans FWD data available to UCB at the time. In some pavement structures a 150-mm (6-inch) aggregate subbase was included. For this layer, a resilient modulus of 165 MPa (24,000 psi) was assumed based on prior experience with Caltrans materials of this type.

The asphalt concrete layer was assumed to have an air-void content of 5 percent. The 20°C stiffness and beam fatigue life relations for the Valley and Coastal asphalt standard mixes were used for the calculations. The Rich Bottom base was assumed to be either 50 mm (2 inches) or 75 mm (3 inches). The air-void content was assumed to be 2 percent and the asphalt content was assumed to be 0.5 percent greater than that determined by the Hveem procedure, based on guidelines presented elsewhere (Table 15). (*12*)

Table 15 Materials properties assumed for Route 710 reconstruction.

Material	Thickness	Material parameters	Poisson's Ratio
Asphalt concrete	Varies	Air-voids: 5 percent, Stiffnesses (MPa): Coastal 2,740 MPa; Valley 8,450	0.35
Rich bottom base	50 mm (2") or 75 mm (3")	Air-voids: 5 percent, Stiffnesses (MPa): Coastal 3,950 MPa; Valley 9,740	0.35
Subgrade	Semi-infinite	M_R = 24, 55, 83 MPa	0.45

The fatigue life was determined using the analysis system used for the other fatigue analyses in this paper. (*11-13*) A design reliability of 95% was selected. For this analysis, tensile strain was calculated at the bottom of both the asphalt concrete layer and the Rich Bottom using the multi-layer elastic theory program CIRCLY. In all cases analyzed, the critical tensile strain was found to occur at the bottom of the Rich Bottom layer. Full bonding was assumed between all pavement layers. The rutting in the unbound layer was determined using the vertical compressive strain at the top of the subgrade and the Asphalt Institute criterion.

The pavement thicknesses for the design ESALs and the two asphalts as determined by the analysis procedure are shown in Table 16. The pavements were first designed to ensure adequate thickness for the fatigue life, however, for the Coastal asphalt the pavement thicknesses were inadequate to resist rutting in the unbound layers due to the lower stiffnesses of the asphalt concrete layer. The thicknesses of the Coastal asphalt pavements were increased until adequate protection was provided to prevent rutting in the unbound layers.

Table 16 Design pavement thicknesses for reconstruction of rigid pavement (design life of 200 million ESALs).

Subgrade Modulus (MPa)	Rich Bottom Thickness (mm)	Valley asphalt (mm)	Coastal Asphalt (mm)
24 (3,500 psi)	0	370	490
	50 (2")	325	460
	75 (3")	320	455
55 (8,000 psi)	0	345	430
	50 (2")	300	405
	75 (3")	295	400
83 (12,000 psi)	0	325	400
	50 (2")	285	375
	75 (3")	280	370

Note: Thicknesses include Rich Bottom layer

A short analysis was performed to evaluate the effect of air-void content on the fatigue life of the pavement for the Valley asphalt mix (Table 17). In this analysis, the air-void content of the asphalt concrete layer was increased from 5 to 6 percent and from 2 to 3 percent for the Rich Bottom base. For this analysis, the subgrade modulus is 24 MPa (3,500 psi) and the design traffic of 200 million ESALs

Table 17 Effect of increased air-void content on required pavement thicknesses (mm).

Rich Bottom Thickness	6 percent air-voids	5 percent air-voids
none	410	370
50	335	325
75	330	320

It can be seen that increasing the air-void content by one percent causes a very large increase in the thickness of the asphalt concrete. It must therefore be emphasized that every effort must be made to achieve the target air-void content in the field so that adequate fatigue performance is ensured.

Conclusions

1. A high priority for Caltrans is the development of rigid and flexible strategies for pavement reconstruction on urban highways in California. Strategies are needed that will meet the requirements for longer life and constructability.
2. A strategy for flexible pavement reconstruction of existing rigid pavements has been developed based on laboratory test results, HVS test results, and mechanistic analyses. The strategy was developed to meet the Caltrans objectives for longer life and constructability. The following elements of the strategy have been verified and calibrated to some degree:
    - Compaction of the asphalt concrete layer to an air-void content of about 5 percent,
    - Inclusion of a “Rich Bottom” at the bottom of the asphalt concrete layer, with enhanced fatigue properties from compaction to about 2 percent air-voids and an asphalt content increased by about 0.5 percent to improve compactability,
    - Use of highly rut resistant mixes in the “critical zone” for rutting, found to be within about 100 to 150 mm of the pavement surface, and
    - Consideration of less conservative design criteria for rutting in the unbound pavement layers.
3. A large difference was found in the predicted performance of mixes with the Coastal and Valley AR-4000 binders.

Recommendations

1. Although substantial work has been done, further validation and calibration using Rich Bottom pavements should be performed. This should primarily include HVS testing of Rich Bottom pavements for rutting and fatigue. The possibility of cracking originating at the surface should be investigated, and better definition of strategies to mitigate its effect on pavement life is needed.
2. In order to reduce the apparent conservatism in current procedures, criteria for rutting of the unbound layers should be investigated for super heavy duty flexible pavements with asphalt concrete layers of 200 mm and thicker. In particular, more HVS tests are needed for wetter environments than have been tested to date. Laboratory tests and analyses are needed to better understand this distress mechanism and to extrapolate the results to field conditions.
3. Better understanding of the critical zone for mix rutting is needed for the different temperature environments in California and different mix properties in order to determine the minimum total asphalt concrete layer thickness required to implement Rich Bottom pavements.
4. Further investigation and HVS and field performance validation is needed of the apparent large differences in fatigue performance between pavements with Valley and Coastal and other asphalt binders.

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