The Phase One I-710 Rehabilitation Project: Initial Design (1999) to Performance After Five Years of Traffic (2008)

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ABSTRACT

The paper contains a summary of information used and developed on the I-710 Long-Life asphalt concrete (AC) Freeway Rehabilitation Project in Long Beach, California – 2.7 miles, six lanes plus shoulders project. It includes the asphalt mix and structural pavement section designs, some construction experience, “lessons learned” from both the design and construction activities associated with the project, and an assessment of pavement performance in the period 2003-2008 (five years under traffic). This project is the first major freeway rehabilitation project in California incorporating 55-hour weekend closures for the construction of a flexible pavement. The rehabilitation includes both *full-depth hot mix asphalt (HMA) sections* which replace the existing plain, jointed portland cement concrete (PCC) pavement under the overcrossings and at interchanges and *HMA overlays* on cracked and seated plain, PCC on the sections between the interchanges and overcrossings.

Results of the asphalt mix and pavement section designs are briefly described. These designs made use of shear and flexural fatigue test data and design methodology developed during the Strategic Highway Research Program (SHRP) and the WesTrack test program technologies as well as results from the California Accelerated Pavement Testing Program (Cal/APT).

Results of measurements of pavement deflections using the Falling Weight Deflectometer (FWD) which were taken at approximately yearly intervals from soon after completion of construction, (Summer 2003) for a period of approximately five years 9to Fall 2008). The FWD deflection results were used to back calculate moduli for both full-depth asphalt concrete sections (6 sections, 3 in each direction) and the asphalt concrete overlays on the cracked and seated PCC (4 sections, 2 in each direction). A summary of the moduli and critical tensile strains in the full depths pavement sections are included. All strains were determined to be less than 70 microstrain..

The project development evaluation are a cooperative effort of Caltrans, Industry, and the University cooperative effort (termed the Long Life Pavement Rehabilitation Flexible Pavement Task Group). Some of the lessons learned from this partnered effort in both the design and construction activities are also briefly summarized since some changes from conventional practice in construction specification and practices were incorporated.

INTRODUCTION

In 1998 the California Department of Transportation (Caltrans) embarked on a Long-Life Pavement Rehabilitation Strategies (LLPRS) program to rebuild approximately 2,800 lane-km (1,740 lane-mi.) of deteriorated freeways in the 78,000 lane-km (48,400 lane-mi.) system of highways under Caltrans jurisdiction. The goals of this program include provision of pavements with design lives of 30+ years with minimal maintenance and utilization of fast-track construction to minimize delays and inconvenience to the traveling public.

The purpose of this paper is to provide a brief summary of various aspects of Phase 1 of the I-710 Freeway Rehabilitation in Long Beach, California, the first of the long life asphalt pavement rehabilitation projects constructed in this corridor These include: asphalt mix and structural pavement section designs; construction experience; and, pavement performance in the period 2003-2008 (5 plus years under traffic). This project is one of the first major freeway rehabilitation projects in the U.S. incorporating 55-hour weekend closures for the construction of long-life asphalt pavements. It includes both *full depth hot mix asphalt (HMA) sections* to replace the existing portland cement concrete (PCC) pavement under the over crossings and at interchanges and *HMA overlays* on cracked and seated PCC on the sections between the interchanges and over crossings.

The project involved a Caltrans, Industry, and University cooperative effort (LLPRS Flexible Pavement Task Group) for: development of HMA mix and pavement design, construction evaluations, and, performance evaluations over a five year period at the completion of construction. The HMA mix and pavement section designs were based on: shear and fatigue test data using equipment developed during the Strategic Highway Research Program (SHRP) (1,2,3) as well as results from the California Accelerated Pavement Testing Program (4,5). The construction specifications included some innovations (6) as did the construction activities during the 8 weekend closures (7). Post construction activities included: interviews by members of the Task Group with both Caltrans District 7 project related staff and Contractor representatives to obtain "lessons learned" from design and construction activities; and, periodic measurements of pavement performance during the 5 plus of first plus years of use, and analyses of these measurements.

Summaries of this information are included in the following sections.

PAVEMENT SITE

Interstate 710, located in Southern California in Los Angeles County, is a heavily trafficked route carrying traffic in and out of the Ports of Long Beach and Los Angeles. Prior to construction, average daily traffic (ADT) was 155,000 vpd during weekdays, with 13 percent trucks. The project is situated between the Pacific Coast Highway (SR 1) and Interstate 405, as shown in Figure 1. This segment of the freeway is about 4.4 km (2.7 mi.) in length with three lanes in each direction (26.3 lane-km [16.3 lane-mi.]), and includes four overpasses.

The original as-designed pavement structural section consisted of 200 mm (8 in.) of plain, jointed portland cement concrete (PCC), 100 mm (4 in.) of cement treated base (CTB), 100 mm (4. in.) of aggregate base (AB and 200 mm (8 in.) of imported subbase material (ASB). Having been constructed during and opened to traffic in 1952 and not overlaid prior to this project, the pavement was in very poor condition. Two rehabilitation strategies were selected:

1. Where there were no overcrossings, and therefore no clearance issues, the existing portland cement concrete (PCC) was cracked and seated and overlaid with HMA (2.8 km [1.7 mi.] total length); and
2. Under the structures where minimum vertical clearance requirements did not allow an overlay, the PCC pavement, CTB, AB and some the ASB were removed. Full-depth HMA sections were utilized with the freeway grade reconstructed and lowered to provide the required clearance for Interstate projects (1.6 km [1.0 mi] total length).

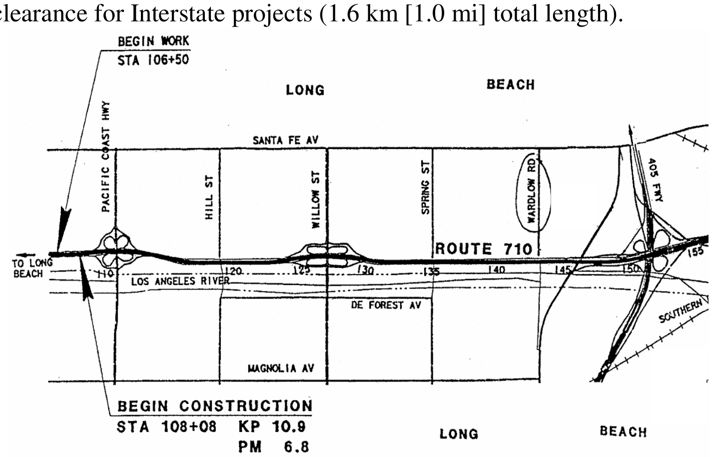


FIGURE 1: Project location.

MIX AND STRUCTURAL SECTION DESIGNS

SHRP mix evaluation technology (1, 3) augmented by research from the Caltrans Partnered Pavement Research Center (PPRC) Program (prior to 2000 the program was called the Cal/APT Program) (4, 5) were used for both mix design and structural pavement section determinations.. The pavement sections were designed to accommodate 200 x 10^6 ESALs, equivalent traffic estimated for a 30-year period by Caltrans District 7 Staff. The frameworks for mix design and pavement analysis are described in References (1) through (3).

Materials

Representative materials which were typically used for asphalt paving in the Los Angeles Basin were supplied by Valero Marketing and Supply Company (formerly Huntway Refining) (asphalt binders) and Vulcan Materials Company, (formerly CAL/MAT) (aggregate), through the Asphalt Pavement Association located in Southern California (APACA).

Two *asphalt binders* were utilized: 1) AR-8000 paving asphalt (AASHTO MP1 designation PG64-16); and 2) polymer-modified asphalt, PBA-6a* (AASHTO MP1 designation PG64-40). The *aggregate* was from the San Gabriel River Valley at Azusa, California.. The aggregate grading used in the mix design and analysis phase is shown in Figure 2 and meets the Caltrans specifications for a dense graded aggregate.

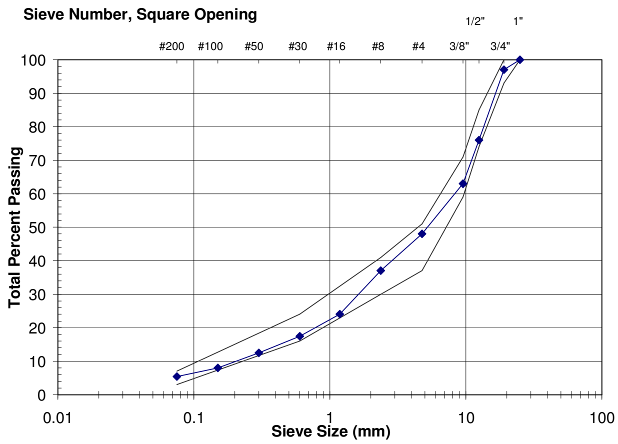


FIGURE 2 Aggregate grading.

Mix Evaluations

A series of mix tests were performed for both mix design and analysis purposes. These included Hveem Stabilometer tests at 60°C (140°F) (CT 367); repeated simples shear tests at constant height (RSST-CH) at 50°C (122°F) and 60°C (140°F) (AASHTO T 320) and flexural fatigue tests at 20°C (68°F) (AASHTO T 321). For the mix with the PBA-6a binder, a limited number of fatigue tests were also performed at 10°, 25°, and 30°C (50°, 77°, and 86°F).

The results of the Stabilometer tests were used to select the range of binders to be used for the RSST-CH, for this mix, 4.2 to 5.2 percent (dry aggregate basis). While tests were performed at both 50° and 60°C (122° and 140°F), the design binder was selected using the results at 50°C since this temperature of is likely close to the critical temperature T_c, for this portion of I-710. A shear stress of 69 KPa (10 psi) was repeatedly applied with a loading time of 0.1 sec and a time interval between load applications of 0.6 sec. This stress and time of loading have been used for both mix analysis and design (e.g.1, 8). Test specimens, 150 mm (6.0 in.) in diameter and 50 mm (2.0 in.) high, were obtained by coring from slabs compacted by rolling wheel compaction. The tests are normally conducted for 5000 stress applications or to a permanent shear strain of 5 percent, whichever occurs first.

For each specimen, test results of permanent shear strain, γ_p, and the number of load repetitions, N, are used to develop an equation of the form:

$$\gamma_p = aN^b \quad \textbf{(1)}$$

The coefficients *a* and *b* of equation (1) are obtained from the adjusted test data using a regression analysis, usually for values of N ≥ 100 or 1000 repetitions depending on mix response. Results of the tests are summarized in Table 1. As seen in this table shear stiffnesses for the PBA-6a* mixes are less than those for mixes containing the AR-8000 binder; yet the resistance to permanent deformation of the PBA-6a* mixes are higher than those for the AR-8000 mixes.

Fatigue tests performed on the AR-8000 mix at 20°C (68°F) were conducted in the controlled-strain mode of loading. Load was applied sinusoidally (no stress reversal) using third point loading at a frequency of 10 Hz (*2*). The test specimens 63.5 mm (2.5 in.) wide by 50 mm

TABLE 1. RSST-CH Test Results at 50C

Binder Type	Binder Content, percent (aggregate basis)	Average Air-Void Content (percent)	N at γ_p = 0.05	G†, MPa (psi)
AR-8000	4.2	4.8	5.08×10^4	74.3 (10.8×10^3)
	4.7	3.6	1.72×10^5	82.0 (11.9×10^3)
	5.2	3.0	2.42×10^4	63.1 (9.14×10^3)
PBA-6a*	4.2	5.5	2.67×10^5	27.3 (3.96×10^3)
	4.7	3.8	1.23×10^6	32.2 (4.65×10^3)
	5.2	5.1	2.26×10^5	26.2 (3.79×10^3)

† *G* measured at *N* = 100 repetitions.

(2 in.) high and approximately 400 mm (16 in.) long were sawed from slabs compacted by rolling wheel compaction. Results of these tests are shown in Figure 3. The tests were performed at two binder contents: 4.7 and 5.2 percent (aggregate basis).

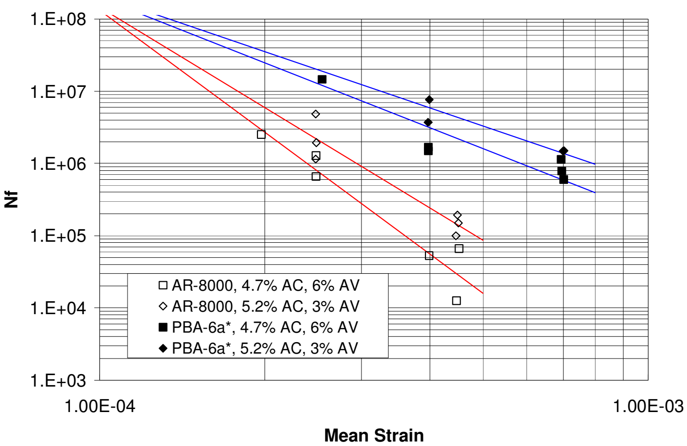


FIGURE 3 Controlled-strain fatigue tests at 20°C (68°F), 10 Hertz frequency.

Mix stiffnesses were determined from both the RSST-CH and flexural fatigue tests. While flexural stiffness was measured only at 20°C (68°F), the shear stiffnesses at 50 and 60C (122°F), F and 140°F) together with the Shell procedure (9) were used to obtain the range of stiffness values shown in Figure 4.

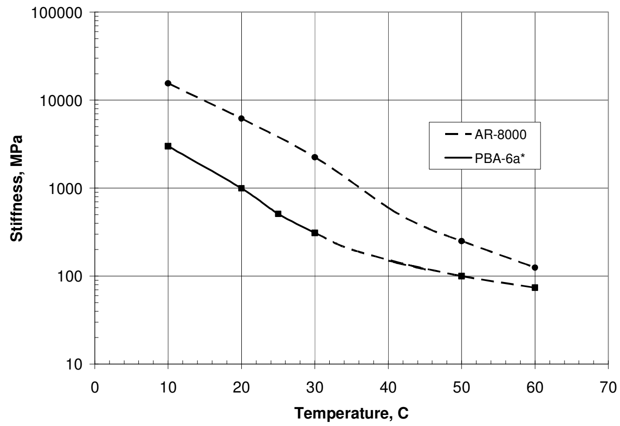


FIGURE 4. Mix stiffness versus temperature; mixes containing AR-8000 and PBA-6a* binders

Mix Designs

Results of the RSST-CH tests were used to determine the design binder content for the surface course mix following the procedure described in Reference (1). The decision was made to base the mix design on traffic expected during the first five years. While the total traffic for the thirty year period was estimated to be 200×10^6 ESALs, a design value of 30×10^6 ESALs was selected considering both current traffic and different estimates of traffic growth. Similarly, the binder content for the AR 8000 mix was selected on the assumption that traffic might be applied to this course prior to surfacing with the PBA 6a* mix for up to one year. Figure 5 shows the results of the shear tests together with traffic levels for the five and one year periods. The levels represented the equivalent laboratory test repetitions of the estimated traffic at the critical temperature, 50°C (122°F) (1,3). Design binder contents of 4.7 percent (aggregate basis) were selected for both mixes considering reasonable tolerances for mix production to insure that

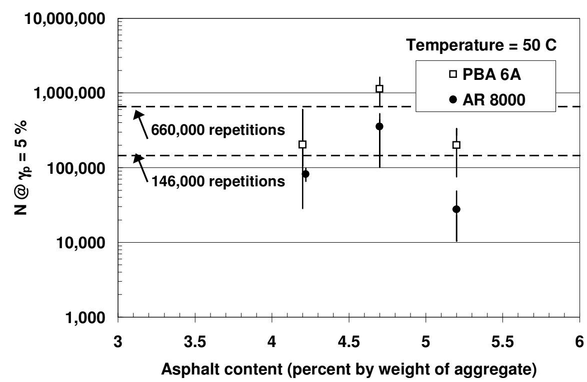


FIGURE 5. Repetitions to 5 percent permanent shear strain, N_{supply}, versus binder content; tests at 50°C (122°F).

the mixes would meet the performance requirements.

To evaluate the mix design prior to construction, an overlay was constructed on an existing plain, jointed PCC pavement at the Richmond Field Station of UC Berkeley. The overlay consisted of 75 mm (3 in.) of the mix with the PBA-6a* binder over 75 mm (3 in.) of the AR-8000 mix, both at 4.7 percent binder content. Industry provided the same materials as used in the mix design from Southern California as well as contractors for mixing and constructing the overlay.

Following construction, a Heavy Vehicle Simulator (HVS) was used to load the PBA-6a mix with about 10,000 (*one-way*) repetitions per day of a 40 kN (9,000 lb.) load on dual tires with a cold inflation tire pressure of 690 kPa (100 psi). The temperature of the pavement was maintained at the critical temperature, 50°C (122°F), at a 50 mm (2 in.) depth.

Results of the accelerated loading on the PBA-6a* mix carried to about 170,000 repetitions (*one-way*) is shown Figure 6. Also shown in the figure are results obtained from an earlier study (10) using both a dense-graded asphalt concrete with AR-4000 asphalt cement (Stabilometer "S" value = 43) and an asphalt rubber gap-graded hot mix (Stabilometer "S" value = 23) both meeting current Caltrans stability requirements for overlay pavements It will be noted that the PBA-6a* mix performed significantly better in terms of rutting than the other two mixes.

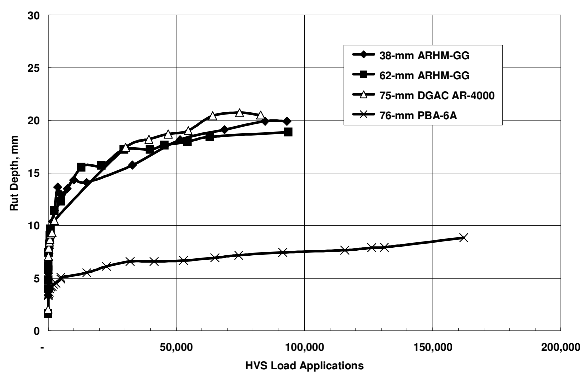


Figure 6. Rut depth versus HVS load applications with 40 kN load on dual tires at 50°C (122°F).

Structural Section Designs

Structural pavements for both the full depth AC replacement section and the overlay section on the cracked and seated PCC pavement were designed as described in References (11, 12).

The approach to structural section design for the full-depth HMA pavement followed that developed during the Strategic Highway Research Program (SHRP) (3) and extended for California conditions through the CAL/APT program based on multilayer elastic analyses (4,5) A number of different sections were analyzed (11).

Thickness of the full depth HMA layer was selected based on resistance to fatigue cracking from tensile strain applications at the bottom of the HMA layer. Fatigue test data shown in Figure 3 were used and the section was designed to accommodate 200 x 10^6 ESALs in the design lane. To insure that the rutting resulting from permanent deformations in the underlying untreated materials does not contribute significantly to surface rutting, the vertical compressive strain at the subgrade surface was limited (13,14). Its determination was also computed using layered elastic analysis (15).

A number of different combinations of materials were evaluated using the above noted methodology. The final section selected consists of: 75 mm (3 in.) of the PBA-6a* mix;150 mm (6 in.) of the AR-8000 mix with 4.7 percent content; and a 75 mm (3 in.) "rich-bottom" layer constructed with the AR-8000 binder at 5.2 percent binder content. The rich bottom layer is used to permit increased compaction and placed in the bottom portion of the full-depth section to improve the fatigue resistance of the pavement. Its location is not expected to affect the rutting resistance of the mix near the surface (4).

Stiffness and fatigue expressions used for the design are shown in TABLES 2 a and 2b used for the design.

TABLE 2a. Material Properties for Pavement Analysis

Material	Modulus MPa (psi)	Poisson's Ratio	Remarks
Subgrade	83 (12,000)	0.45	"reasonable" design value; determination from back calculation, FWD tests
	55 (8,000)	0.45	lower bound based on backcalculations
AR-8000 mix	6372 (0.924×10^6)	0.35	20°C, 4.7% binder content, Vair = 5.6%
	6898 (1.0×10^6)	0.35	20°C 5.2% binder content, Vair = 3.2%
PBA-6a mix	1008 (0.146×10^6)	0.35	20°C, 4.7% binder content, Vair = 5.2%
	918 (0.133×10^6)	0.35	20°C, 5.2% binder content, Vair = 3.3%

TABLE 2b. Fatigue Characteristics of Evaluated Mixes*; 20°C (122°F)

Mix Binder	Binder Content –percent	Fatigue Equation	Remarks
AR-8000	4.7	$N_f = 5.142\times10^{-15}\varepsilon_t^{-5.602}$	$\varepsilon_t > 70\times10^{-6}$ mm/mm
	5.2	$N_f = 5.083\times10^{-11}\varepsilon_t^{-4.614}$	$\varepsilon_t > 70\times10^{-6}$ mm/mm
PBA-6a	4.7	$N_f = 2.229\times10^{-4}\varepsilon_t^{-2.989}$	$\varepsilon_t > 70\times10^{-6}$ mm/mm
	5.2	$N_f = 9.478\times10^{-3}\varepsilon_t^{-2.589}$	$\varepsilon_t > 70\times10^{-6}$ mm/mm

To accommodate the 200×10^6 ESALs, the structural sections resulting from the fatigue analyses produced computed strains less than 200×10^{-6} mm/mm 9in./in.). Accordingly it was necessary to extrapolate the laboratory fatigue data to the range of the lower computed strains (Figure 3). In addition, it has been assumed that at strain values of less than 70×10^{-6} mm/mm (in./in.) the likelihood of fatigue failure is small(16).

The Asphalt Institute subgrade strain criteria were used to minimize rutting contributed by the unbound layers (14), i.e.,

$$N = 1.05\times10^{-9}\varepsilon_v^{-4.484} \qquad \textbf{(2)}$$

Where: ε_v is the vertical compressive strain at the top of the subgrade and ESALs of 50×10^6 were used to satisfy the subgrade strain requirement (16).

For the final structural pavement section resulting from the analyses, the PBA-6a* mix was recommended for use only in the upper part of the structure even though its fatigue resistance is higher than mixes containing the AR-8000 binder. To satisfy the subgrade strain criteria a substantially thicker section consisting of all PBA-6a mix would be required since its stiffness is only about one-sixth that of the AR-8000 mix at 20°C. The 75 mm (3 in.) thickness of the rich bottom layer is based on a limited analysis reported in Reference (4).

It is important to emphasize that the resulting design is based on the assumption that the PBA-6a* and AR-8000 mixes (with 4.7 percent binder) will be compacted to an air-void content of 6 percent and that the rich-bottom AR-8000 mix (with 5.2 percent binder) to 3 percent. Though these requirements are more stringent than those used by Caltrans in 1999, the construction specifications were prepared to reflect these compaction requirements.

The section includes a 25 mm (1 in.) porous friction course using an asphalt-rubber binder placed on the surface of the PBA-6a* mix. This layer, in addition to reducing hydroplaning, tire splash, and noise, is intended to reduce wear on the PBA-6a* mix. It is intended that this mix will be periodically removed and replaced during the design life of the structure.

Design of the overlay pavement structure required a different approach than that used for the full-depth HMA replacement structure (12). In this case, the primary concern was to select an adequate thickness to mitigate the loss in pavement serviceability resulting from *reflection cracking*. Current Caltrans practice for this type of construction has consisted of an asphalt concrete overlay on top of an asphalt-saturated non-woven fabric interlayer placed on an AC leveling course on the cracked and seated concrete. Thicknesses of AC overlays of this type of the order of 125-150 mm (5-6 in.) have provided service lives of the order of 10 years (10 to 20 $x10^6$ ESALs). With existing practice as a guide, the same concept was used for the overlay pavement structure. The problem then became one of determining a "reasonable" thickness to sustain 200×10^6 ESALs.

The approach taken was to perform finite element (FE) simulations on idealized representations of asphalt concrete overlays of different thicknesses on the cracked and seated 200 mm (8 in.) thick plain, jointed PCC pavement. In the analyses, crack spacing obtained by the pavement cracking procedure was assumed to average about 0.91 m (3.0 ft.). Reference (12) contains a detailed discussion of the various alternatives considered and associated analyses.

For design of this type of overlay, the effects of both traffic loading and environmentally induced stresses must be considered, the latter resulting from length changes caused primarily by temperature changes. By breaking the concrete slabs into smaller sections, the relative movements at the cracks/joints from temperature changes are reduced. In this case, the effects of temperature induced deformations/stresses were neglected based on temperature changes estimated to occur at a depth of 229 mm (9.0 in.), the actual depth of the final overlay section.

The NIKE 2D FE program was selected to determine the effects of traffic loading (12). Details of the analyses and the basis for selection of the final overlay thickness are Atdescribed in Reference (12). The resulting thickness consists of: 25 mm (1 in.) porous friction course using an asphalt-rubber binder; 75 mm (3 in.) of the PBA-6a* mix; 96.5 mm (3.8 in.) of the AR-8000 mix with 4.7 percent content; asphalt saturated fabric as an interlayer; and 30.5 mm (1.2 in.) leveling course with the same AR-8000 mix at 4.7 percent. This design has the advantage that the 75 mm (3 in.) PBA-6a mix and the 25 mm (1 in.) open-graded layer can be placed continuously since both the full depth and crack-seat and overlay sections have upper most layers that are the same.

At the junction of the replacement section and the overlay, the thickness of the replacement section is 432 mm (17 in.).

Construction Considerations

For successful performance of these pavement structures, strict attention to pavement construction was required, necessitating careful control of the mix components, mix compaction, and layer thickness.

In a significant departure from Caltrans "Standard" requirements, the specifications included mix design "recommendations" with mix performance requirements. This allowed the contractor to optimize the materials selection process while requiring the specified mix performance characteristics.

Prior to construction, shear and fatigue test data were required to be submitted by the contractor to Caltrans for mix approval. This requirement was incorporated into the project specifications to ensure that the mixes to be used by the contractor met the performance characteristics shown in Table 6 (6). The performance characteristics specified correspond to those used in the mix and pavement design processes. In addition, prior to construction, materials were required to be submitted to Caltrans for verification of these mix characteristics.

TABLE 3. Asphalt Concrete Mixture Performance Requirements [Table 39-3A from Reference (6)]

Design Parameters		Test Method	Requirement
Permanent Deformation (min.)	PBA-6a* (modified)[2]	AASTHO TP7-94 modified[1]	660,000 stress repetitions[3,4]
	AR-8000[2]	AASTHO TP7-94 modified[1]	132,000 stress repetitions[3,4]
Fatigue (min.)	PBA-6a* (modified)[5,6]	AASTHO TP8-94 modified[1]	7,000,000 repetitions[4,8] 60,000,000 repetitions[4,9]
	AR-8000[5,7]	AASTHO TP8-94 modified[1]	300,000 repetitions[4,8] 15,000,000 repetitions[4,9]

Notes to Table 4.1:

1. Included in the testing guide provided upon request.
2. At proposed asphalt binder content and with mix compacted to 3 ±0.3% air-void content.
3. In repeated simple shear test at constant height (RSST-CH) at a temperature of 50C
4. Mean of 3 specimens
5. At proposed asphalt binder content and with mix compacted to 6 ±0.3% air-void content (determined using AASHTO 209 (Method A)).

6. At proposed asphalt binder content, minimum stiffness at 20C and a 10-Hz load frequency must be equal to or greater than 150,000 psi (1000 MPa). At proposed asphalt binder content, minimum stiffness at 30C and a 10-Hz loading frequency must be equal to or greater than 45,000 psi (300 MPa).
7. At proposed asphalt binder content and 6 ±0.3% laboratory air-void content (determined using AASHTO 209 (Method A)), minimum stiffness at 20C and a 10-Hz loading frequency must be equal to or greater than 900,000 psi (6200 MPa). At proposed asphalt binder content plus 0.5 percent and 3 ±0.3% laboratory air-void content (determined using AASHTO 209 (Method A)), minimum stiffness at 20C and a 10-Hz loading frequency must be equal to or greater than 990,000 psi (6800 MPa).
8. At 300 × 10-6 mm/mm. Results shall be reported for this strain level, but may be obtained by extrapolation. Minimum number of repetitions required prior to extrapolation defined within test procedure.
9. At 150 × 10-6 mm/mm. Results shall be reported for this strain level, but may be obtained by extrapolation. Minimum number of repetitions required prior to extrapolation defined within test procedure.

Compaction and other quality control requirements were increased. Mixes containing the PBA-6a* mix and the AR-8000 at a binder content of 4.7 percent were to be compacted to an air-void content of about 6 percent (93–97 percent of the TMD and compaction requirements for the rich-bottom mix were an air-void content of not more than 3 percent.

Prior to this project, Caltrans practice did not require a tack coat between lifts for multiple lift construction if placed in the same construction day and without trafficking. For the I-710 project, this practice was changed to require a tack coat between each lift. This change in practice resulted from observations of the performance of HVS test sections as part of the CAL/APT program (1).

Construction

Details of the construction activities associated with the I-710 project are contained in Reference (7). This section briefly summarizes these activities to provide an indication of the approaches followed.

Construction of the project was accomplished in six construction stages. In the first stage, the median was widened and the old metal beam guardrails were replaced with concrete barriers. The second stage included excavating, widening, and paving the outside shoulders up to the existing pavement surface elevation. The remaining four stages involved the main work of rehabilitating the four full-depth asphalt concrete (FDAC) sections under the overpasses and the two AC overlays of the cracked and seated PCC (CSOL) sections. In the Caltrans plan, a total of 10 consecutive weekend closures were scheduled for completion of these last four stages.

Encouraged by the incentives ($100,000 per weekend less than 10), the contractor revised the Caltrans staging plan by splitting the freeway into eight segments in order to complete those segments in eight weekend closures, as shown in Figure 7

During the main rehabilitation work, the contractor was required to adopt a “counter-flow traffic” closure strategy. With this strategy, one direction of the freeway is closed for the construction work zone (CWZ). Traffic is then rerouted through crossover areas located at both ends of the CWZ to two lanes in each direction (three traffic lanes plus the shoulder) on the other half of the freeway. The two directions of traffic are separated by a moveable concrete barrier (MCB). During the first four weekend closures, the contractor shut down the southbound side for construction while maintaining two lanes of traffic in each direction on the northbound side. The closure was reversed during the latter four weekend closures. A short (8-hour) full closure of the entire freeway was used for mobilization and demobilization in each 55-hour weekend closure.

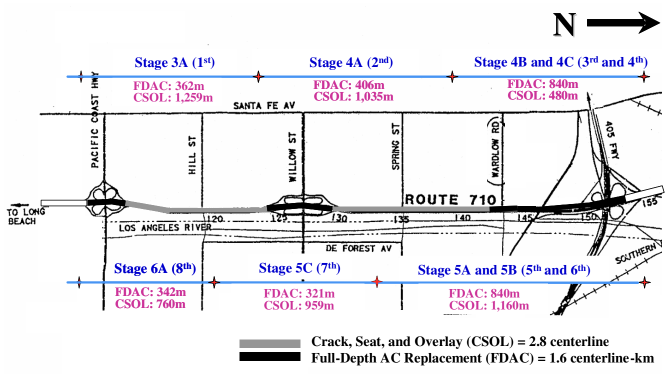


FIGURE 7. Contractor's revised staging plan (sequence of 55-hour weekend closures shown in parentheses).

The sequence of activities for the major CSOL and FDAC construction for a typical 55-hour weekend closure as well as the contractor's overall CPM schedule submitted for a typical 55-hour weekend closure are described in Reference (17).

Relative to the construction planning, a program called *CA4PRS*, developed by PRC staff with support of Caltrans, WSDOT, MNDOT, and TXDOT (7), assisted both the Caltrans and the Contractor in planning for the construction operations described in the previous section. The program was further calibrated by monitoring three of the eight weekend closures and is now available for use on future projects. Associated with this program is also a traffic simulation program to assess the impact of the closure on traffic operations of the network in the area surrounding the construction zone (17).

Lessons Learned Following Construction, Fall 2003

On November 4, 2003, a team consisting of Caltrans, Industry, and University members of the Flexible LLPR Task Group, conducted post-construction interviews with Caltrans staff for one-half day (morning) and Contractor and Materials Suppliers for one-half day (afternoon). The purpose of the interviews was to ascertain what went well with the project and aspects that should be considered for improvement on future projects of this type.

Results of the interviews, which included considerations of *design, construction,* and *traffic monitoring* as well as *other factors,* are summarized in Reference (17). These results are presented in tabular form by the following categories: what went well; and what didn't go well required improvement). In some instances both groups noted the same point; other issues were noted either by Caltrans or Contractor Staff. Also, a summary of general recommendations resulting from the interviews is included in Reference (17) with the intent of assisting in planning for future projects of this type.

A fewof the 'what went well' activities by both Caltrans and Contractor were: 1) contingency plan was used by contractor; 2) use of any of the three different mixes during weekend closures presented no difficulties; and, 3) traffic monitoring plan. Some of what didn't go well noted by both groups included:1) development of mix designs by the contractor using shear and fatigue tests; 2) differences in as-designed and as-built layer thicknesses in dig out areas; and, 3) QC/QA dealing with large quantities of HMA in short time period (~14,000 tonnes during closures). Also, in the dig out areas lack of information on location of underground utilities, subsurface conditions (e.g, ground water), core data, etc. were problems.

A few of the comments and recommendations were: 1) pre-bid conference between all parties involved in the project should be mandatory; 2) 55 hour weekend closures satisfactory for this type of project; 3) tests should be done in advance of construction in the areas where the full depth HMA sections and a geothechnical engineer should be on site at the time of construction to deal with any unforeseen problems in these areas; and 4) human resources were "stretched"- accordingly successive closures should be limited to three to five consecutive closures with one or two weekends intervening.

In addition to the "lessons learned", two additional points should be noted. The first is the matter of "partnering." In this instance, new technology (SHRP developed) was used for mix evaluation. Had there been partnering on a technical level at the outset of the project between Caltrans staff, the Contractor, and members of the LLPRS Flexible Pavement Task Group regard the use of the shear and fatigue tests, this problem possibly would have been minimized. Thus an important conclusion from this experience is that technical partnering at the outset of a project of this type is mandatory.

The second point is related to the matter of the establishment of criteria like those shown in Table 4. These criteria required the performance of tests for which there was little precision and bias information. Thus, when establishing new criteria of this type, consideration must be given to requirements based on the best available statistical information related to the test procedures utilized. Such information would also assist in mitigating the problem referred to in the previous paragraph. This was done for Phase of the 2 of the I-710 rehabilitation (17).

Follow-up Investigations

To track the performance of the pavement sections, a plan for heavy weight deflectometer (FWD) testing was instituted. Testing was carried out at approximately yearly intervals for a period of 5 years. Other post-construction activities have included in-situ sampling to obtain beam fatigue and RSST-CH tests on specimens obtained from a few sections of the pavement for comparison with the initial design test results (included in Reference [17]).Ride quality was measured on two occasions, i.e., April 2006 and January 2009. The January 2009 measurements were conducted on both the Phase 1 Rehabilitation and the remaining section of the freeway from I-405 to I-10. Skid measurements were also performed in February 2006.

The detailed results of the FWD tests and their interpretations are included in Reference (17) and will only briefly discussed in this section. Also included are brief summaries of ride quality, defined by the International Roughness Index (IRI), and the skid measurements.

The schedule for the HWD tests throughout the 5 plus years after construction was as follows: 1)November 2003; 2) September 2004; 3) September 2005; 4) December 2005, northbound; February,2006, southbound; 5) September 2007, Lanes 2 and 3 only; 6) September 2008. The schematic for testing all three lanes is shown in Figure 8

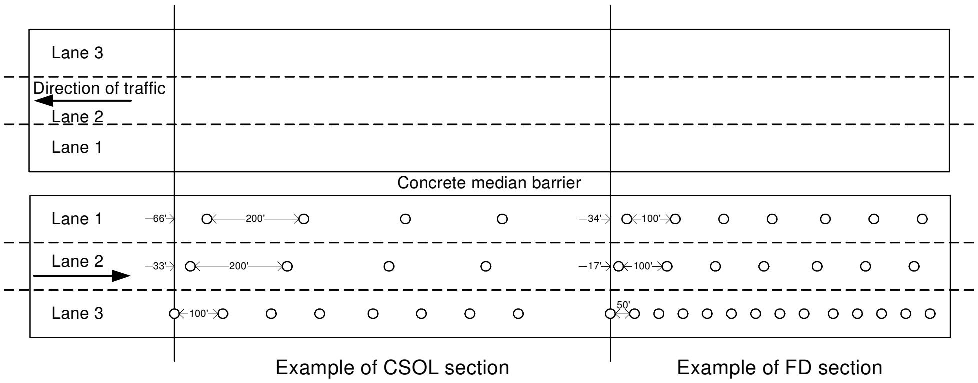


FIGURE 8. Deflection testingz patterns.

The layout of the project, Figure 1 and the sequence of construction, Figure 7 assisted in establishing the Sections for the HWD testing, labeled 1 through 5, beginning at the Pacific Coast highway (PCH). Section numbers increase in the northbound direction (i.e to the I-405 Freeway). Sections 1, 3, and 5 are the full-depth HMA sections and Sections 2 and 4 are the HMA overlay p on the cracked and seated PCC sections. This resulted in a total of ten sections, five northbound and five southbound.

HWD tests were conducted in the outer wheel path of each of the lanes (except for the September 2007 test program). Testing was normally accomplished between the hours of 9 PM and 7 AM. Three load level were applied at each point; target values were 40, 80, and 120 kN (9, 18 and 27 kips). Measured deflections and back-calculated stiffness moduli have been used to illustrate pavement response over the five-years of trafficking. The back-calculation program used to determine the stiffness moduli of the individual layers in each of the sections is termed CalBack, developed by Dr. Per Ullidtz of Dynatest (one the UC PRC research partners in the Caltrans supported program).

Deflections from the 120 kN load measured in Lane 3 in both the north and southbound directions are shown in Figures 9 and 10 for the five year period.

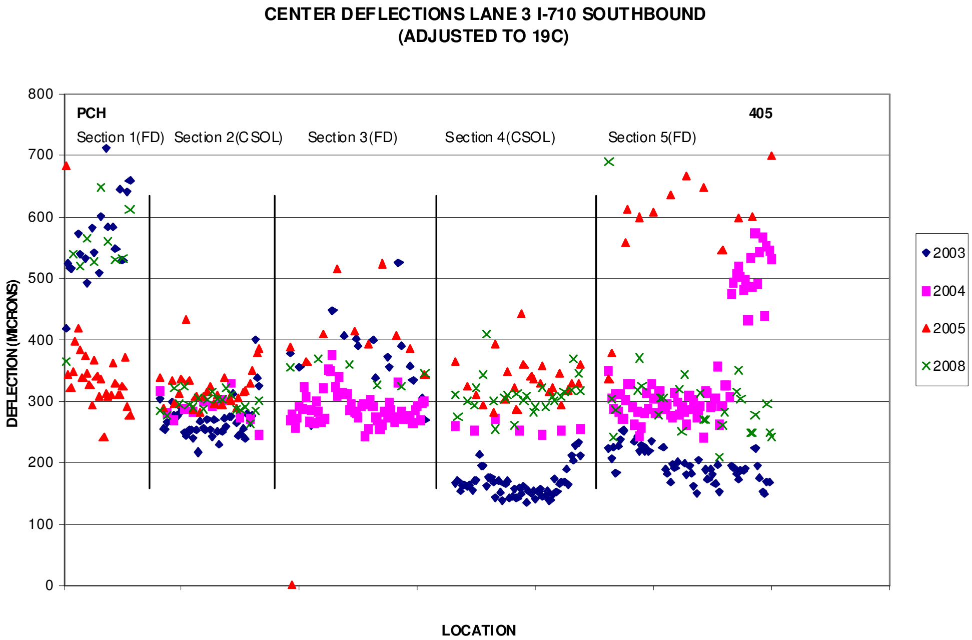


FIGURE 9: Deflections, Lane 3 Southbound, 2003 to 2008, 19°C (66°F).

It will be noted that deflections are highest in Section 1 southbound. This was the first section constructed. The 150 mm (6 in.) granular base and engineering fabric were not used although they had been called for to insure that the rich bottom AC layer would have a good working platform to achieve the requisite degree of compaction. Instead, a layer of HMA approximately 50 mm (2 in. thick) was placed directly on the sand subgrade but was not sufficient to permit proper compaction of the rich bottom layer (resulted in one of the lessons learned recommendations noted above).

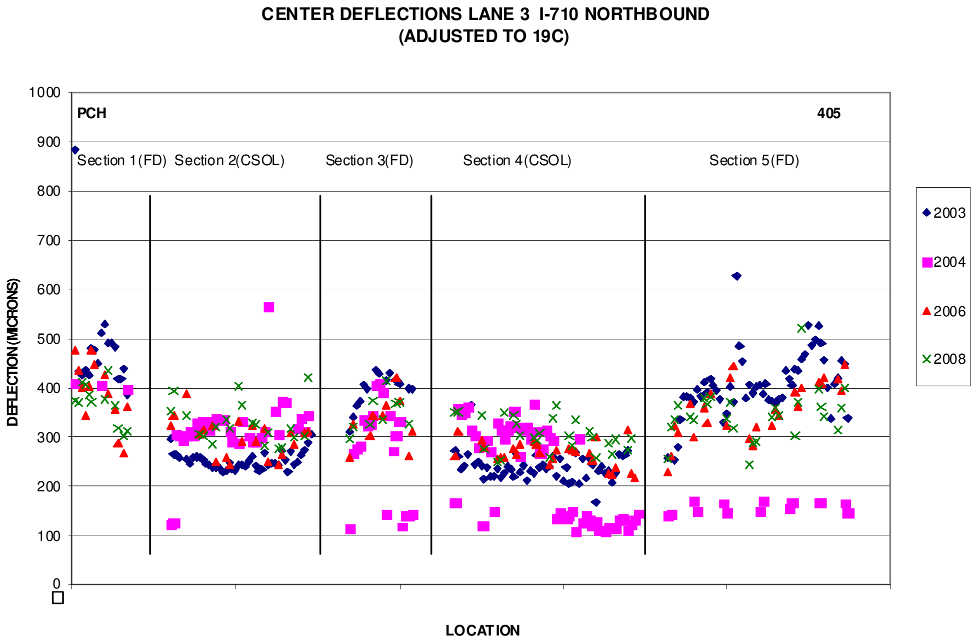
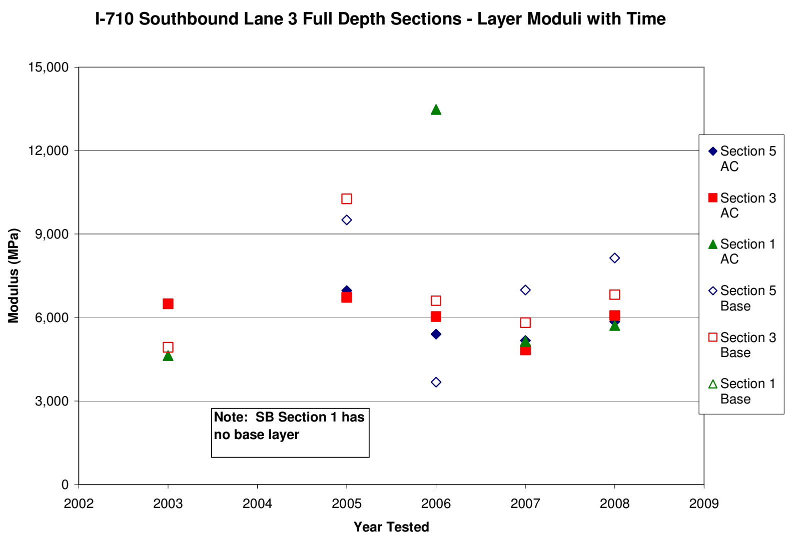


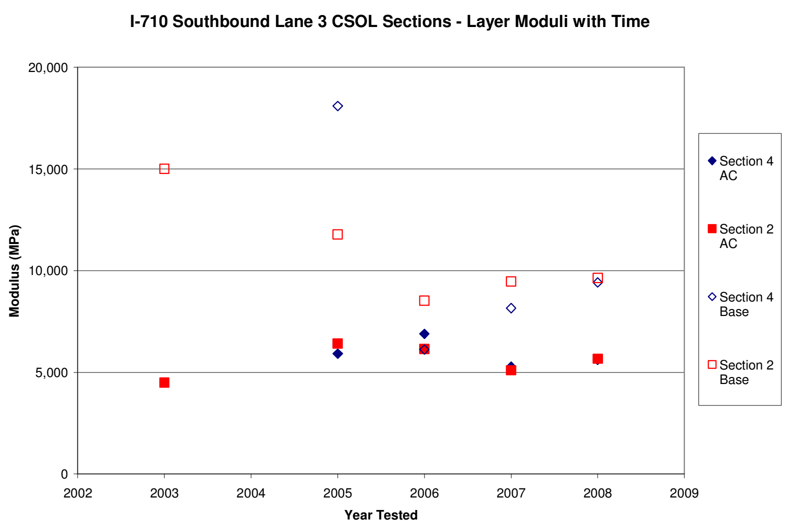
FIGURE 10. Deflections, Lane 3 Northbound, 2003 to 2008, 19°C (66°F).

In both figures it will be noted that deflections in the cracked and seated sections are less than those in the full depth AC sections. It will also be noted that the deflections measured in 2008 are in the same range as those measured in earlier years. Comparisons of the deflections for the corresponding north and southbound sections, other than Section 1 southbound, are observed to be similar.

A comparison of the deflections for the 40, 80, and 120 kN (9, 18 and 27 kips)loads indicated a linear relationship between load and deflection. Accordingly the measured deflections for the 120kN (27 kips) load were used for the back calculation of layer moduli. Computed moduli based on the 2008 deflection data for AC and base for Lane 3 southbound and northbound are shown in Figures 11 and 12.

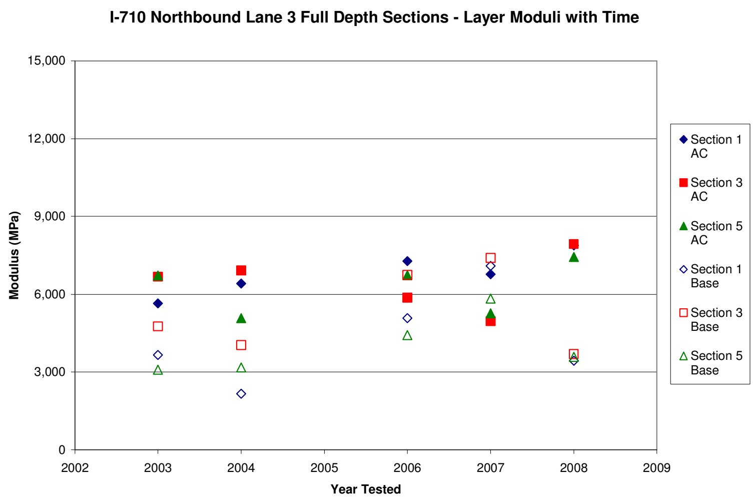


a. Full-depth sections

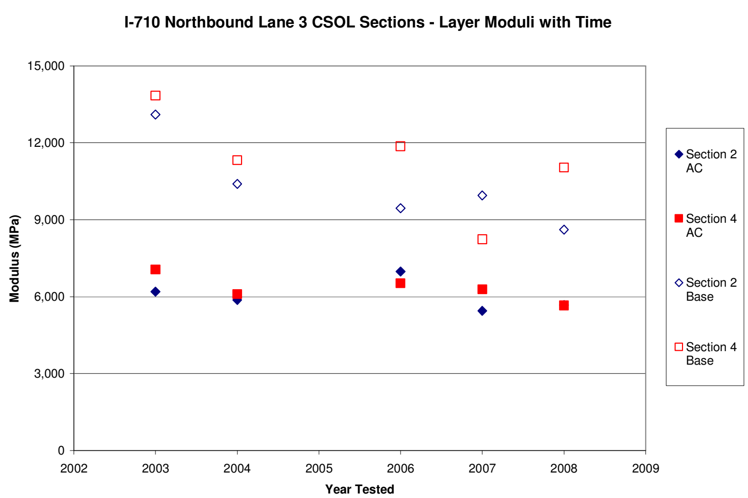


b. Crack, seat, and overlay sections

FIGURE 11. Southbound Lane 3 Layer moduli versus time.



a. Full-depth sections



b. Crack, seat and overlay sections

FIGURE 12. Northbound Lane 3 layer moduli versus time.

From Figures 11 and 12 it will be observed that the back calculated moduli for the AC layers are reasonably uniform in both the southbound and northbound directions. For the three full depth sections the computed moduli average is approximately 7500 MPa (1,100,000 psi) northbound and 6000 MPa (870,00o psi) southbound. For the four (two in each direction) crack, seat, and overlay sections the computed moduli average about 5600 MPa (815,000 psi). This difference in stiffness between the moduli of the crack, seat and overlay and full depth sections is reasonable. Referring to Figure 4, it will be noted that the flexural stiffness of the PBA 6a* mix is about one-sixth of the stiffness AR 8000 mix. The full depth sections have 225 mm (9 in.) of the AR 8000 mix and 75 mm (3 in.) of the PBA 6a* mix whereas the crack seat and overlay sections have 125 mm (5 in.) of the AR 8000 mix and 75 mm (3 in.) of the PBA 6a* mix. The difference in stiffnesses between the northbound and southbound full depth sections could result, in part, from the back calculation procedure.

Computed subgrade stiffness values range from 100 to 150 Mpa (14,500 to 21,800 psi). The aggregate base for the full depth sections consisted of crushed concrete. Computed stiffness moduli were in the range 3000. to 7000 MPa (435,000 to1,000,000 psi). These values are larger than normally associated with untreated aggregate base suggesting some cementing in this material. (A separate study of this phenomenon is underway)

To estimate the pavement performance of Lane 3, both northbound and southbound, for the full-depth sections, multilayer elastic analyses were performed. Back calculated layer moduli for the 2008 FWD tests were used. Selected measures of performance parameters from these back calculations are summarized Table 5.

TABLE 5. Tensile and vertical compressive strains in full depth sections

Section	North bound		Southbound	
	tensile strain, $\varepsilon_t x10^{-6}$	vertical compressive subgrade strain, $e_v x10^{-6}$	tensile strain, $\varepsilon_t x10^{-6}$	vertical compressive subgrade strain, $e_v x10^{-6}{}_t$
1	18	80	49	151
3	17	78	11	82
5	16	72	8.4	73

It will be noted that the strains are low suggesting little damage from fatigue cracking and minimal contributions to permanent deformation from the untreated components of the full depth sections in these sections at this time. However, the results in Table 5 do illustrate a difference between southbound Section 1 and the five other full depth sections because of the absence of the untreated aggregate base in this section.

Longitudinal Profile and Skid Measurements

Longitudinal profile measurements were made in 2006 and 2009 and IRI values were determined. Skid measurements were obtained by Caltrans in 2006.

The longitudinal profile measurements in January 2009 were conducted by the UCPRC throughout the length of the I-710 freeway from the Pacific Coast Highway (SR 1) to the I-10 (Santa Monica) freeway, a distance of approximately 20 miles. Results of these measurements in terms of the International Roughness Index (IRR, in. per mi.) are shown in Figure 13. The first approximately 2.7 miles are the rehabilitated section discussed herein. Table 6 summarizes the results by pavement type, lane and direction.

TABLE 6. IRI by pavement type and direction.

Pavement Type	Overall IRI	Station (mi)	IRI (in/mi)		
			Lane	Northbound	Southbound
HMA-O	89	0.00 - 2.76	1	70	69
			2	90	86
			3	126	91
			average	*96*	*82*
PCC	163	2.76 - 20.5	1	155	151
			2	154	153
			3	199	166
			average	*170*	*157*

Skid measurements were performed by Caltrans in February 2006 using their skid test equipment. The data for the Phase 1 rehabilitation have been summarized and an average skid number, SN_{40}, have been determined for each direction. The average values are: northbound, 47 (126 values), range 38-54; and, southbound, 49 (97 values), range 43-57.

Summary

The I-710 project provided an opportunity to implement approaches for asphalt concrete mix design and structural pavement design developed during SHRP.

The selected pavement section for the full-depth AC replacement structure consists of: 1) a rut-resistant surface course using a mix containing a 100 percent crushed aggregate and PBA-6a* modified binder for both rutting resistance and improved durability characteristics; 2) an asphalt concrete base course containing an AR-8000 asphalt which provides good stiffness characteristics for the structural section; 3) the same mix with 0.5 percent more asphalt and termed a “rich-bottom mix” to provide improved fatigue resistance in the lower part of the structural section; and 4) an open-graded asphalt rubber porous friction course for splash, spray, and hydroplaning resistance and noise reduction as well as providing a protective layer for the PBA-6a* mix. It is anticipated that this surface course will be replaced during the design life of the structure.

The structural section thickness was selected using a procedure developed during SHRP and calibrated to California conditions using the HVS.

Mix designs for both the PBA-6a* and AR-8000 mixes were performed using the SHRP developed simple shear test. The PBA-6a mix was subjected to approximately 170,000 repetitions with the HVS at a pavement temperature controlled at 50°C (122°F) at a depth of 50 mm (2 in.). Results of the test suggest that the PBA-6a* mix should satisfactorily carry the anticipated traffic without excessive deformations.

The pavement structures for the cracked and seated portion of the freeway followed Caltrans practice in that it consists of a leveling course placed on the PCC pavement, followed by an application of an asphalt saturated fabric serving as a membrane interlayer and additional asphalt concrete and the porous friction course. Since the anticipated traffic is significantly greater than that on which current thicknesses are based (i.e., 200×10^6 versus approximately

20×10^6 ESALs), finite-element analyses were conducted for a range in the AC thicknesses. Based on these analyses, the overall thickness of the overlay was increased from about 150 mm (6 in.) to 225 mm (9 in.). The mixes used are the same as those for the full-depth AC replacement structure resulting in continuity of the mixes throughout the project.

Successful performance of the pavement structures requires strict attention to pavement construction including careful control of the mix components during production and the degree of densification during compaction. Results of the QC/QA testing (not included herein) indicate that this objective was achieved.

The "lessons learned" provides worthwhile information to both agency and contractor staff interested in this type of construction. The matter of "partnering" is extremely important to achieve the highest quality possible in the finished pavement structures; high quality is mandatory to achieve performance for the longer time periods associated with this type of pavement rehabilitation.

The constructability program, CA4PRS, served as a useful tool to both the Caltrans and Contractor staff in achieving the necessary goals associated with the 55-hour weekend closure. In this regard, it is interesting to note that the program can provide an indication of the maximum length of full-depth AC that can be placed in dig-out areas in the 55-hour closure. For this project, the maximum thickness of AC for a 3 lane freeway for a 1000-ft. length would be about 20 in., paving only in the traveled way. Similarly, for a 4 lane freeway, the maximum thickness for the same length would be about 15 in.

Following completion of construction in June 2003 a five year program was established to evaluate the performance of the pavement. This program included HWD measurements at approximately yearly intervals through Fall 2008. In addition, some cores and slabs were obtained to assess in-situ mix behavior using performance tests that were used to establish mix designs and the thickness of both the full depth and overlays on the cracked and seated PCC. Longitudinal profile measurements were made in January 2009 on both the rehabilitated sections (Pacific Coast Highway to I-405) and the remaining section (I-405 to I-10). Noise measurements using OBSI equipment were made in 2006 and 2008.

From the HWD measurements moduli of the pavement layers were estimated and performance measures were calculated. The same parameters as used for the original suggest that the pavement is performing as expected. Visual condition surveys conducted in November 2008 indicated some areas with surface distress in the RAC-O surface course. However the satisfactory performance of the pavement to date was corroborated by Caltrans HQ Maintenance) in an e-mail to Dr. John Harvey in early February 2009.

> "In speaking with Leo [Mahserelli] concerning the 710 LL HMA pavement, it's clear that there is no widespread deterioration which would constitute a failure for the project. D7 Maintenance Engineering South Region Engineer, Shay Banduk, sent his assessment and photographs of localized distress, small patches, isolated ravelling and some mechanical damage/gouges but no deterioration which would spell pavement failure. The rehabilitation of the ramps throughout the project were deleted from the project (costs?) and they do look worn, rough and crack sealed/damaged.
>
> However, with no record of exceptional workload or work expenditures from this pavement, and visual inspection by our very experienced pavement reviewer as well as the Region engineer, it would seem to be a case of

mistaken reporting of a pavement failure.

The objective evaluation of pavement condition since construction is our PCS which, as I stated Thursday, reported only light raveling (it is 8 years old) and good to fair IRI numbers".

From the results reported herein thus far, considering the strains determined from the back calculation procedure as well the condition surveys, the information would appear to validate the approach adopted by the LLPRS Flexible Pavement Task Group for this project.

In conclusion, it must be emphasized that the process used to arrive at the designs as well as the construction requirements was a "partnered" effort between Caltrans, the Asphalt Industry in California, and academia through UC Berkeley. This partnering provided an excellent opportunity to successfully implement new ideas and research results on this challenging project in which some of the traditional approaches were insufficient.

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