ARIZONA DEPARTMENT OF TRANSPORTATION

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# THREE YEAR EVALUATION OF I-40 CRACK AND SEAT EXPERIMENTAL PROJECT

**Interim Report**

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**16. Abstract**
In 1986, Project I-40-3(31) was rehabilitated using crack and seat techniques and overlaying with a 4-inch HMAC overlay. The crack and seat technique was utilized to prevent reflection cracking in the HMAC overlay caused by joints in the existing JPCP. The 4-inch overlay was provided to improve the ride quality and to increase the structural capacity of the pavement section. The project was designated as experimental due to ADOT's limited experience with crack and seat techniques.

To evaluate the effect of crack spacing on overlay performance, three crack patterns were tested; 2 ft. by 2 ft., 3 ft. by 3 ft., and 4 ft. by 6 ft. The majority of the 6.5 mile project was constructed using the 3 ft. by 3 ft. spacing. Only two test sections, one tenth of a mile long each, were constructed using the other spacings.

Visual crack surveys and FWD testing were performed at three locations over a period of three years. Roughness, skid, and cracking were obtained as part of ADOT's annual PMS inventory. The results from the FWD testing were used to backcalculate the layer moduli.

Large variations in the effective modulus of the cracked and seated JPCP were obtained using the back-calculation technique. The average moduli typically ranged between 3.3 and 7 million p.s.i.. The effective moduli tended to fluctuate over time, but no definite trends were established. Structural layer coefficients determined by AASHTO and NAPA procedures indicated no decrease with time, except for the modified NAPA computed 4 ft. by 6 ft. coefficient which indicated a slight decrease with time.

The crack and seat technique and HMAC overlay strategy were successful in reducing roughness on this section of Interstate. The eastbound roughness was reduced 254 inches/mile while the westbound roughness was reduced 165 inches/mile. These strategies were not successful in mitigating reflection cracking. Approximately 55% of the joints/cracking have reflected through.

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# SI* (MODERN METRIC) CONVERSION FACTORS

## APPROXIMATE CONVERSIONS TO SI UNITS

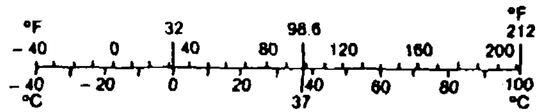
| Symbol | When You Know | Multiply By | To Find | Symbol |
|---|---|---|---|---|
| | **LENGTH** | | | |
| in | inches | 25.4 | millimetres | mm |
| ft | feet | 0.305 | metres | m |
| yd | yards | 0.914 | metres | m |
| mi | miles | 1.61 | kilometres | km |
| | **AREA** | | | |
| in$^2$ | square inches | 645.2 | millimetres squared | mm$^2$ |
| ft$^2$ | square feet | 0.093 | metres squared | m$^2$ |
| yd$^2$ | square yards | 0.836 | metres squared | m$^2$ |
| ac | acres | 0.405 | hectares | ha |
| mi$^2$ | square miles | 2.59 | kilometres squared | km$^2$ |
| | **VOLUME** | | | |
| fl oz | fluid ounces | 29.57 | millilitres | mL |
| gal | gallons | 3.785 | litres | L |
| ft$^3$ | cubic feet | 0.028 | metres cubed | m$^3$ |
| yd$^3$ | cubic yards | 0.765 | metres cubed | m$^3$ |

NOTE: Volumes greater than 1000 L shall be shown in m$^3$.

| Symbol | When You Know | Multiply By | To Find | Symbol |
|---|---|---|---|---|
| | **MASS** | | | |
| oz | ounces | 28.35 | grams | g |
| lb | pounds | 0.454 | kilograms | kg |
| T | short tons (2000 lb) | 0.907 | megagrams | Mg |
| | **TEMPERATURE (exact)** | | | |
| °F | Fahrenheit temperature | 5(F-32)/9 | Celcius temperature | °C |

## APPROXIMATE CONVERSIONS FROM SI UNITS

| Symbol | When You Know | Multiply By | To Find | Symbol |
|---|---|---|---|---|
| | **LENGTH** | | | |
| mm | millimetres | 0.039 | inches | in |
| m | metres | 3.28 | feet | ft |
| m | metres | 1.09 | yards | yd |
| km | kilometres | 0.621 | miles | mi |
| | **AREA** | | | |
| mm$^2$ | millimetres squared | 0.0016 | square inches | in$^2$ |
| m$^2$ | metres squared | 10.764 | square feet | ft$^2$ |
| ha | hectares | 2.47 | acres | ac |
| km$^2$ | kilometres squared | 0.386 | square miles | mi$^2$ |
| | **VOLUME** | | | |
| mL | millilitres | 0.034 | fluid ounces | fl oz |
| L | litres | 0.264 | gallons | gal |
| m$^3$ | metres cubed | 35.315 | cubic feet | ft$^3$ |
| m$^3$ | metres cubed | 1.308 | cubic yards | yd$^3$ |
| | **MASS** | | | |
| g | grams | 0.035 | ounces | oz |
| kg | kilograms | 2.205 | pounds | lb |
| Mg | megagrams | 1.102 | short tons (2000 lb) | T |
| | **TEMPERATURE (exact)** | | | |
| °C | Celcius temperature | 1.8C + 32 | Fahrenheit temperature | °F |




* SI is the symbol for the International System of Measurement

(Revised April 1989)

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## INTRODUCTION

### Background

Interstate 40 is one of Arizona's major east-west trucking routes. It is a four lane divided highway located in the northern portion of the state. Approximately 4% of its 718 miles (total, both directions) consists of plain jointed concrete pavements (JPCP). Although the oldest concrete section was constructed in 1946, the existing concrete roadways were constructed between 1967 and 1972. The total Vehicle Miles Traveled (VMT) on I-40 was 3.75 million in 1987. VMT on the concrete pavement sections amounts to 7.2% of the total VMT on I-40.

As I-40 concrete pavements attain their 20 year design lives they have typically experienced more traffic loading than their design traffic and presently require extensive rehabilitation. One form of rehabilitation utilized is cracking and seating followed by an asphalt concrete overlay. Although the oldest I-40 concrete pavement section was cracked and seated in the early 1970s, this technology has had little use in Arizona since then.

In 1986, project I-40-3(31) was designed for rehabilitation with the crack and seat construction prior to the placement of a 4-inch Hot Mix Asphalt Concrete (HMAC) overlay. This strategy was selected because of the faulting and material deterioration of the existing concrete pavement. Experience across the United States at that time indicated varied results with crack and seat techniques. Several states had tried the procedure and believed it to be ineffective. Other states used it on a routine basis. Due to the contradictory experience and ADOT's limited usage, the FHWA required that the crack and seat projects be classified as experimental. Additionally, they requested that several crack patterns be investigated. This resulted in the development of the test sections on project IR-40-3(59). Two test sections and three crack spacing intervals were incorporated into the project. The objective was to study both the effectiveness of the crack and seat strategy and the effect of crack spacing on the mitigation of reflective cracking.

## PROJECT DESCRIPTION

### Existing Pavement Section

The original concrete construction project extended from milepost 152.1 to 158.6 and included both eastbound and westbound roadways. Each roadway consisted of two 12-ft concrete lanes with 4-ft and 10-ft asphalt concrete shoulders on the inside and outside, respectively. The pavement has a cross slope of 0.015 ft/ft to the outside shoulder and incurs approximately 12,000 AADT with 38% commercial vehicles.

The pavement section consisted of 8 inches JPCP on 4 inches CTB over 6 inches of subgrade seal as shown in Figure 1. The existing EB roadway was opened to traffic in 1968, while the WB was opened in 1967. The eastbound and westbound roadways were constructed from different aggregate sources as evident in color differences in the concrete. Transverse joints were established at 15 ft intervals and skewed 1 to 6 counterclockwise. Both transverse and longitudinal joints were sawed at the time of construction.

In January and April of 1986, District forces resealed the pavement joints and cracks prior to the construction project. The sealing was performed with an ASTM 3405 sealant.

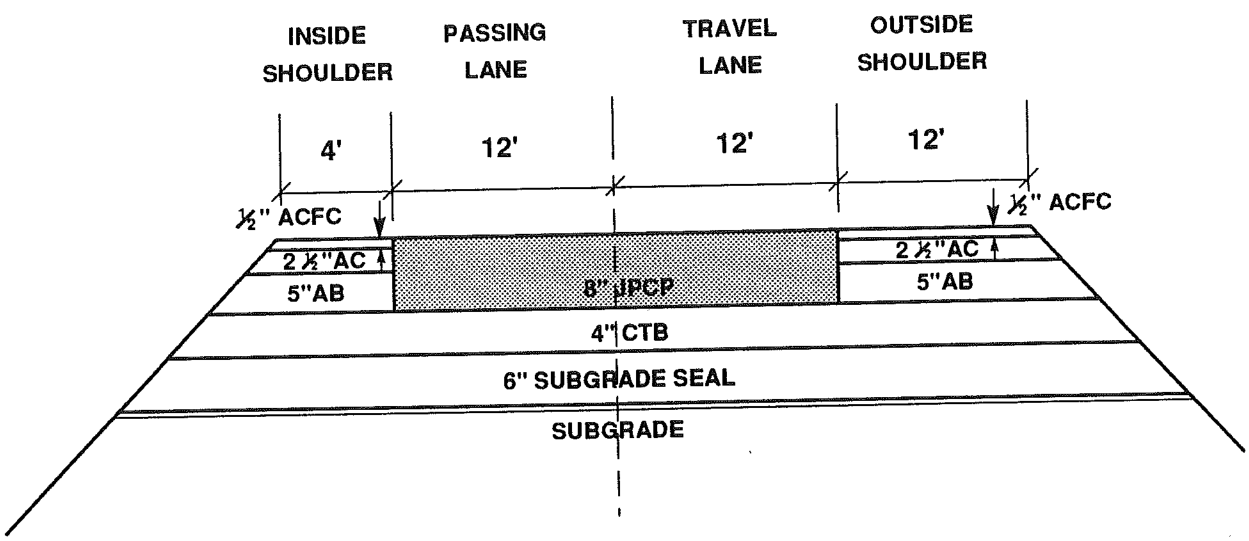



Figure 1- Original Pavement Section

**Design Consideration and Layout**

Two test sections were established on this project to evaluate the effect of crack spacing. The two test sections, each one-tenth of a mile in length, were constructed at the extreme west end of the project as shown in Figure 2. The test sections were constructed at this location to minimize the impact to the construction process. Since the remainder of the westbound roadway and all of the eastbound roadway were to be cracked and seated with a 3 ft by 3 ft spacing, it was more convenient to locate the test sections at one end.

The crack and seat spacings for the test sections were selected as 2 ft by 2 ft and 4 ft by 6 ft. These spacings represented crack intervals which were both larger and smaller than the originally specified 3 ft by 3 ft spacing. The 3 ft spacing was considered the control section. A section which did not utilize cracking and seating (a valid control section) was not utilized. Therefore, a direct comparison of the effectiveness of the crack and seat strategy versus a simple overlay strategy is not possible. The 2 ft by 2 ft and 4 ft by 6 ft spacings represented intervals previously utilized in other states.

### Construction Features

The project consisted of cracking and seating the existing 8 inch JPCP, installing edge drains adjacent to the outside shoulder, and placing a 4 inch HMAC overlay across the entire roadway, including both shoulders. Figure 3 indicates the pavement cross section for the crack and seat project.

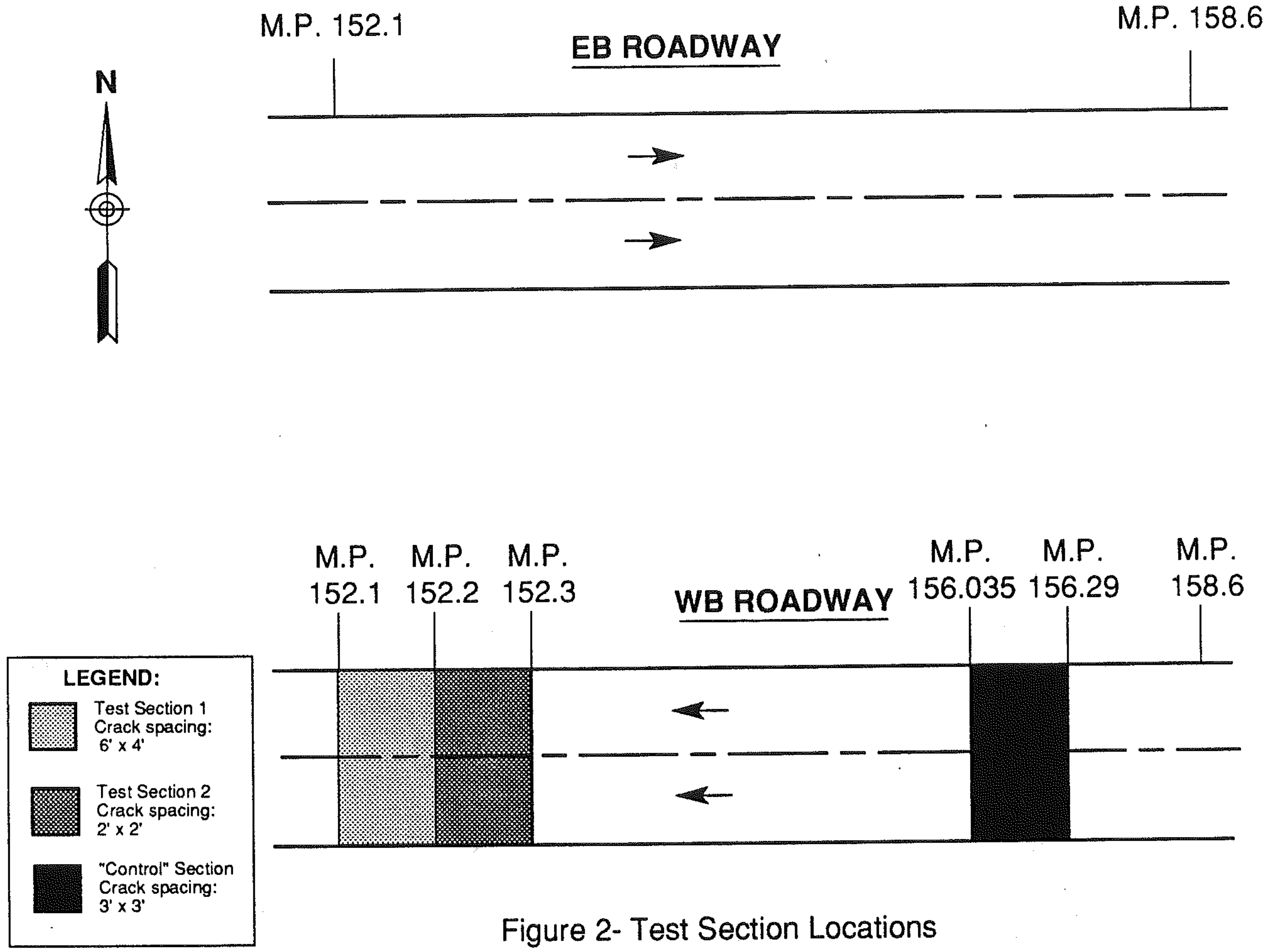



Figure 2- Test Section Locations

The cracking required that the specified spacing would be obtained by a method which produced full depth, generally transverse, hairline cracks at a nominal longitudinal spacing of 3 ft by 3 ft, the only exception was at MP 152.1 to 152.2 (WB) and at MP 152.2 to 153.3 (WB) where the nominal spacing was 4 ft by 6 ft and 2 ft by 2 ft, respectively. Inspection of the cracking effectiveness was performed by applying water to a test section once each day. A Michigan Whip Hammer was used to perform the cracking operation.

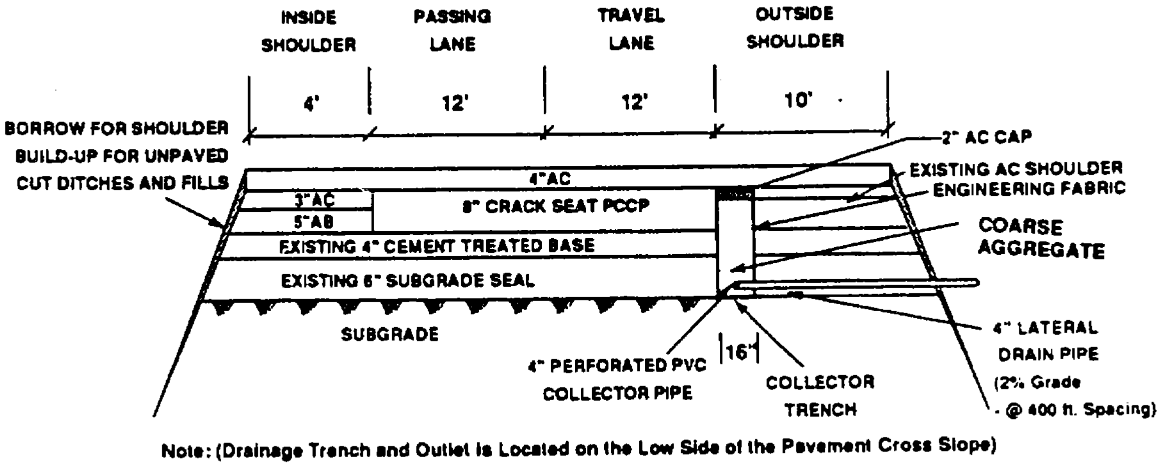



Figure 3- Typical section of the cracked and seated pavement with overlay

Construction specifications required seating to be accomplished by at least two passes of a 50-ton pneumatic roller or until the concrete pieces were assured of being seated. During construction, this was accomplished by a "wagon-like" tire roller fitted with sand ballast. Cracked pavement segments were required to be seated not more than 24 hours prior to receiving the asphalt concrete overlay and not more than 72 hours after cracking. Traffic was allowed on the cracked and seated concrete for no more than 72 hours. Evaluation of the seating effectiveness was based upon the judgement of the Engineer.

The drainage trench was constructed after the crack and seat operation and prior to overlay placement. The trench was installed at the edge of the JPCP shoulder to a depth of 18 inches. A perforated collector pipe was utilized within the granular trench backfill. Lateral drain pipes were placed every 400 ft. The drainage trench was typically located adjacent to the outside lane since this is the lowest point in the pavement section. In areas which had high superelevations the trench drains were located at the inside shoulder.

The 4 inch overlay was placed in two 2 inch lifts of 1/2 inch dense graded AC. The first lift was placed as a leveling course and the second as a surface course. Both lifts utilized AC-30 in the mix and as a tack coat. During placement of the leveling course compaction problems were experienced. The required density was not obtained for all lots. Additionally, bumps occurred in the leveling course at random locations over joints in the underlying JPCP. This apparently was due to a reaction between the joint filler material and the hot mix. The bumps were milled off prior to placement of the surface course. During placement of the surface course compaction problems were again experienced. The required density was not obtained for all lots.[1]

## PROJECT LOCATION

The project is located approximately 10 miles west of Williams in north central Arizona. It is situated within the Kaibab National Forest at an average elevation of 5900 ft. Vegetation in the area consists predominantly of Juniper trees and grass. Soils in this area are residual soils derived from volcanic parent rock and range from well graded sand (SW) to clays (CH).

The project is situated within Climatological Zone 8 of the ADOT Preliminary Engineering Manual.[2] Mean annual precipitation is approximately 20 inches. The mean daily temperature is 60° F with extremes between 97° F and -22° F. The freezing index in this location is 700.

The thirty year average annual high and low temperature is shown in Figure 4.[3] The meteorological data obtained during and since construction is superimposed on this plot. As noted in the figure, weather during and since construction has been typical for the area.

Figure 5 indicates a histogram of the daily mean temperature difference since construction. The maximum mean temperature difference was 32° F and the 80th percentile value was 29° F.

The referenced meteorological data was recorded at the National Weather Services Station at Williams approximately 10 miles from the project site.

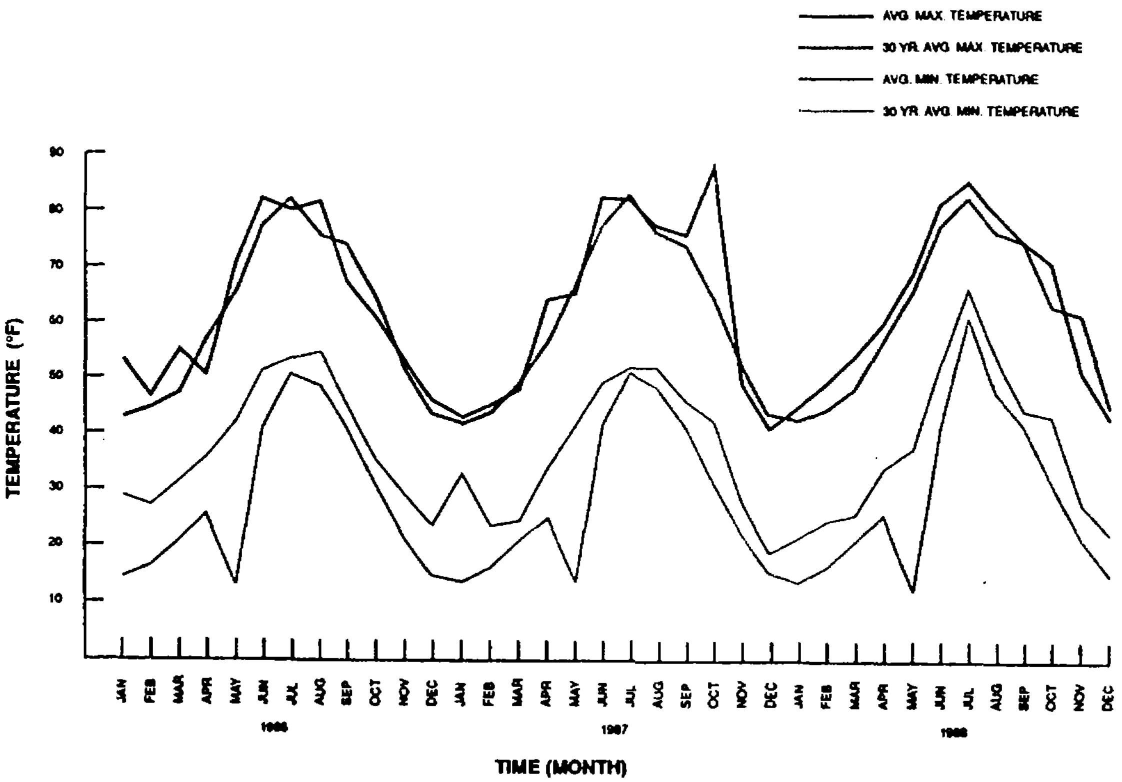



Figure 4- Thirty Year Average High and Low Temperatures

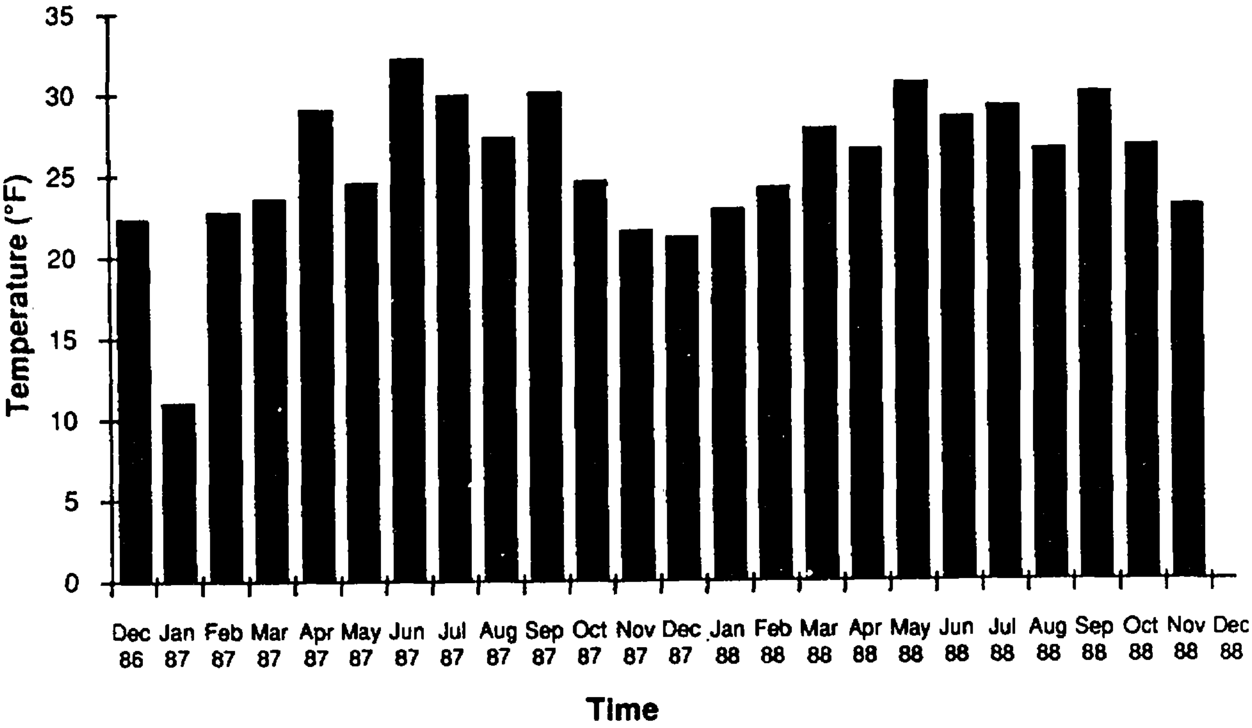



Figure 5 - Daily Mean Temperature difference on the Project Since Construction

## EXISTING PAVEMENT PERFORMANCE

### Traffic History

Traffic on this section has increased at an average rate of 3.3% per year. The 1984 ADT was 8300 with 24% commercial vehicles. Figure 6 indicates estimated ESALs since construction of the JPCP. These data are shown since 1968 even though the westbound roadway was opened in 1967.

### Subgrade Conditions

During the design phase, eight subgrade samples, four from each roadway direction, were retrieved to characterize the quality of the subgrade. R-value tests were conducted as well as Atterberg Limits and Moisture Density relationships. The results of these tests are shown in Table 1.

These test results indicate considerable variability in the subgrade properties, ranging from essentially non plastic, well graded sand (SW) to medium to high plasticity, clays (CH). The subgrade conditions exhibited moisture contents typically 3-12% below optimum. It should be noted that at MP 155.9 EB and MP 153.0 WB the in situ moisture content was above optimum. The moisture content at MP 155.9 is especially notable since it is essentially non-plastic with only 18% passing the number 200 sieve size.

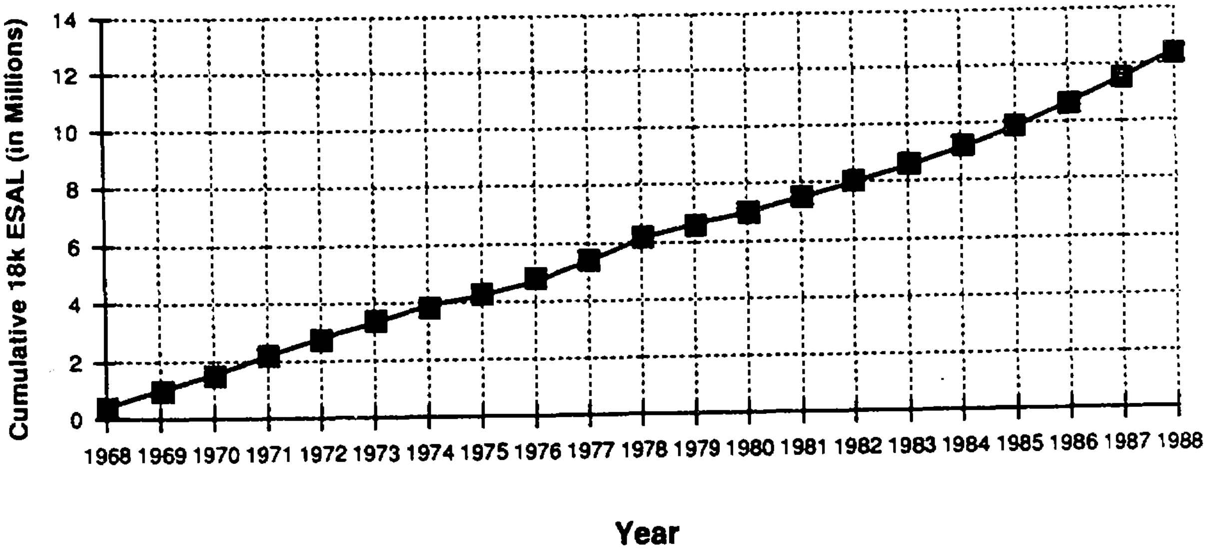



Figure 6- Traffic on the Crack and Seat Project since Original Constuction

The subgrade variability is not uncommon for soils in the vicinity of this project which are of volcanic origin. However, a six inch subgrade seal was utilized in the original construction and it is not known whether these test results characterize the subgrade seal, the subgrade soils beneath the seal, or a combination of both.

TABLE 1 - SUBGRADE TEST RESULTS

| Location | R-value | P.I. | %Passing #200 | Moisture Content Optimum | Moisture Content In-place | k* (pci) |
|---|---|---|---|---|---|---|
| 152.3 EB | 32 | 21 | 36.7 | 21.8 | 9.8 | 238 |
| 153.3 EB | 75 | 1 | 10.1 | 18.4 | 8.9 | 1048 |
| 155.9 EB | 68 | 1 | 18.3 | 20.5 | 29.0 | 823 |
| 158.2 EB | 45 | 19 | 22.7 | 24.0 | 21.5 | 372 |
| 159.0 WB | 58 | 16 | 17.4 | - | 8.8 | 600 |
| 156.5 WB | 80 | 4 | 10.6 | 17.6 | 13.4 | 1250 |
| 155.0 WB | 34 | 14 | 28.3 | 21.0 | 12.6 | 260 |
| 153.0 WB | 12 | 34 | 80.3 | 29.6 | 30.6 | 150 |

*Calculated from ADOT correlation between R-value and k.

## FUNCTIONAL PERFORMANCE

### Testing Performed and Purpose

The functional performance of the roadway can best be described by its serviceability. In this report the present serviceability is described in terms of roughness, measured by the Mays Ride Meter, and frictional characteristics assessed using the Mu-Meter.

Arizona performs an annual inventory of its highway network and records this information on a route-milepost basis. Roughness, skid, patching, and faulting are measured in the travel lane and entered into the pavement management system database.

Roughness is determined by a Mays Ride Meter (car) traveling at 50 MPH which obtains continuous readings between mileposts. The readings are summarized in inches per mile and the results assigned to the milepost location at which the readings begin. Once the field data is obtained it is normalized to 1971 calibration values to provide consistency with time.

Skid (friction) is determined by a Mu-Meter which is a continuous recording friction measuring trailer. Continuous readings are obtained for a five hundred foot section of wet pavement starting at a milepost location. The readings are averaged and assigned to the milepost.

**Roughness Data**

Roughness data as a function of time since construction are shown in Figures 7 and 8 for each roadway direction and milepost location. These data indicate that the various milepost locations have performed similarly, increasing in roughness approximately 10 to 11 inches per year. Linear regressions were performed on these data to establish the trend of roughness increase. The results of these analyses are shown in Table 2. The estimated roughness at original construction has been taken as the value of the 1972 data since it was the first year that Mays Meter measurements were available. Milepost 152 and 158 data were dropped from the analysis because of contaminated data due to only partial contribution of JPCP to the roughness measurement. Roughness measurements obtained from the asphalt concrete pavements adjacent to each end of the project are included in the measurements recorded at MP 152 and MP 158.

The results shown in Table 2 indicate that the estimated initial Mays Roughness for the WB direction ranged between 91 inches/mile and 128 inches/mile with an average of 106 inches/mile. The rate of increase varied between 6.7 and 14.1 inches/mile/year with an average of 10.1 inches/mile/year. The average Mays roughness just prior to rehabilitation (1986) was 255 inches/mile. The initial roughness for the EB direction ranged between 98 inches/mile and 220 inches/mile with an average of 148 inches/mile. The rate of increase varied between 7.9 and 16 inches/mile/year with an average of 11.2 inches/mile/year. The average Mays Roughness just prior to the rehabilitation (1986) was 318 inches/mile.

**Skid Data**

Skid data as a function of time for each roadway direction and milepost location are shown in Figure 9. Regression equations were developed for each of the milepost locations for the years 1980-1985. The results are shown in Table 3. All the milepost locations have performed similarly. Regression results show that the average rate of skid decrease was 0.3 and 2 Mu-meter units per year for the EB and WB directions, respectively. Milepost 152 data was not included in the analysis. The PMS Mu-meter data collected at this location includes only AC pavement.

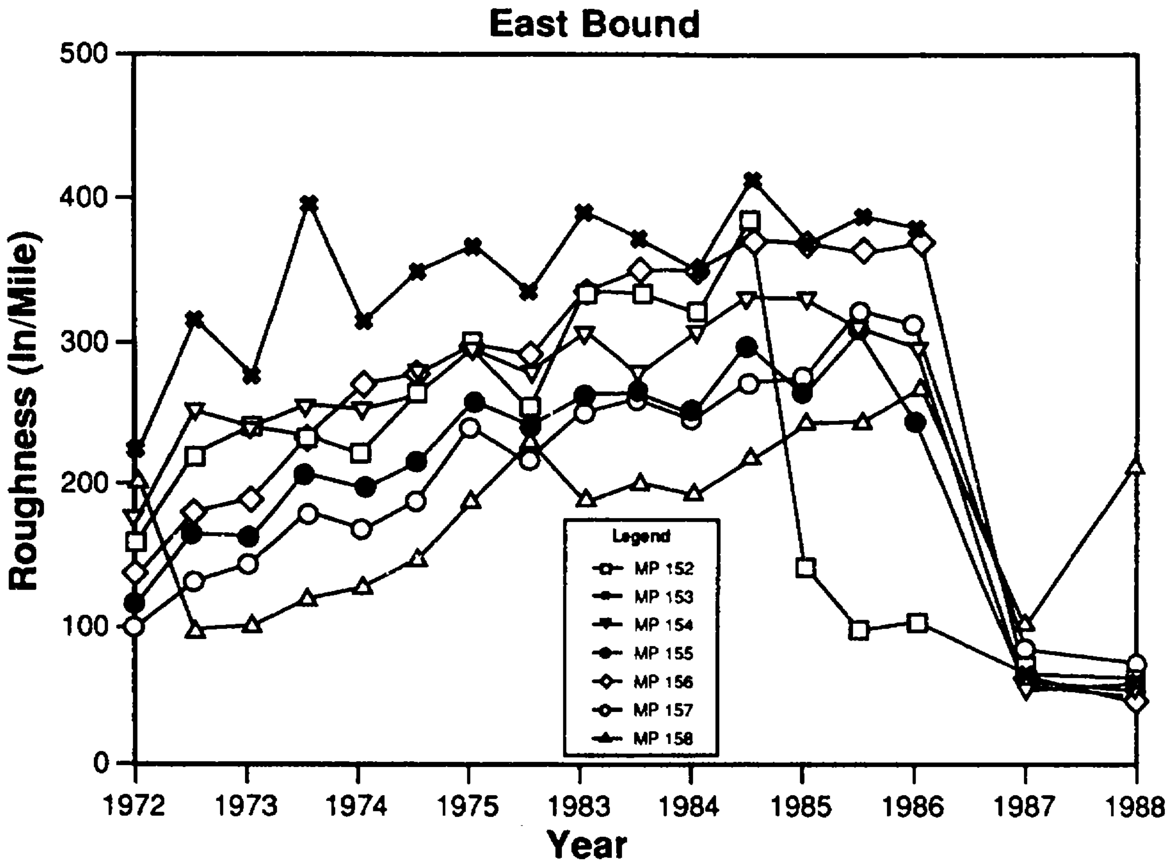


Figure 7- Mays Meter Roughness vs. Time

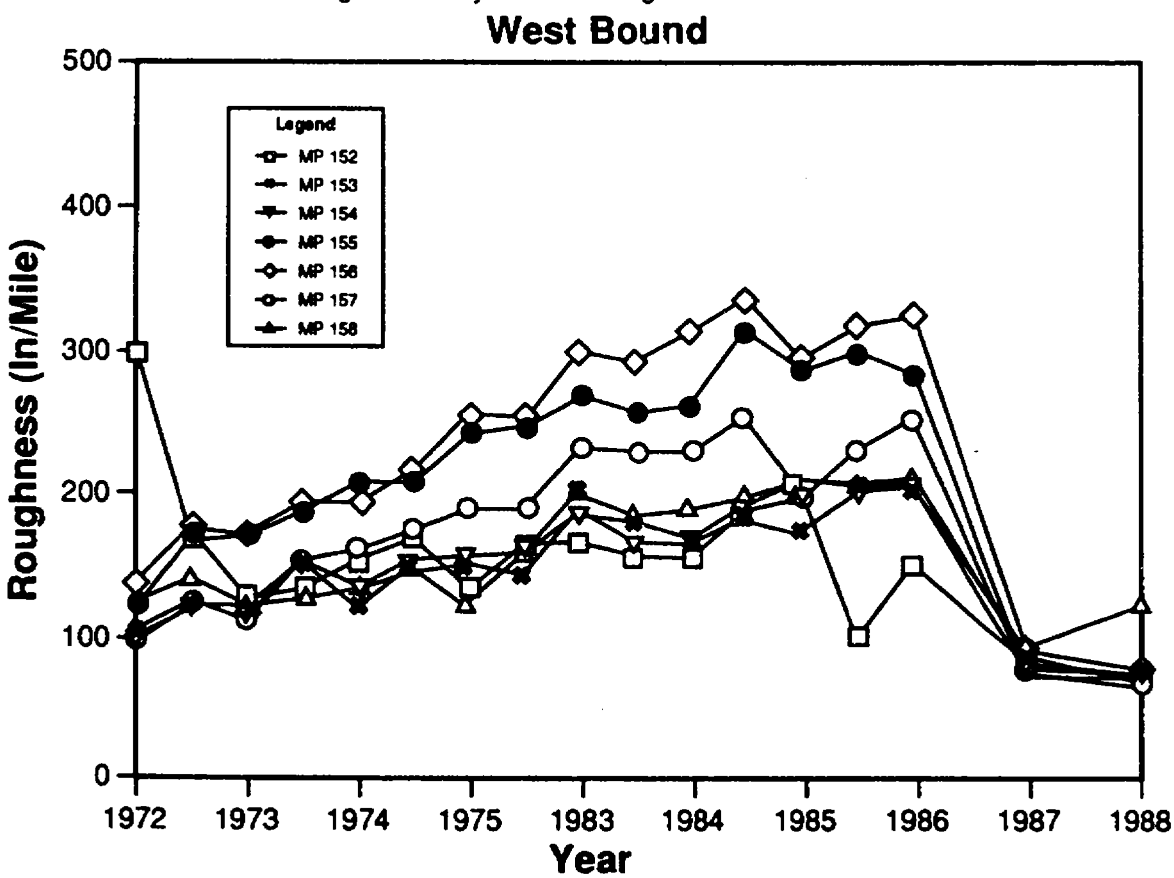


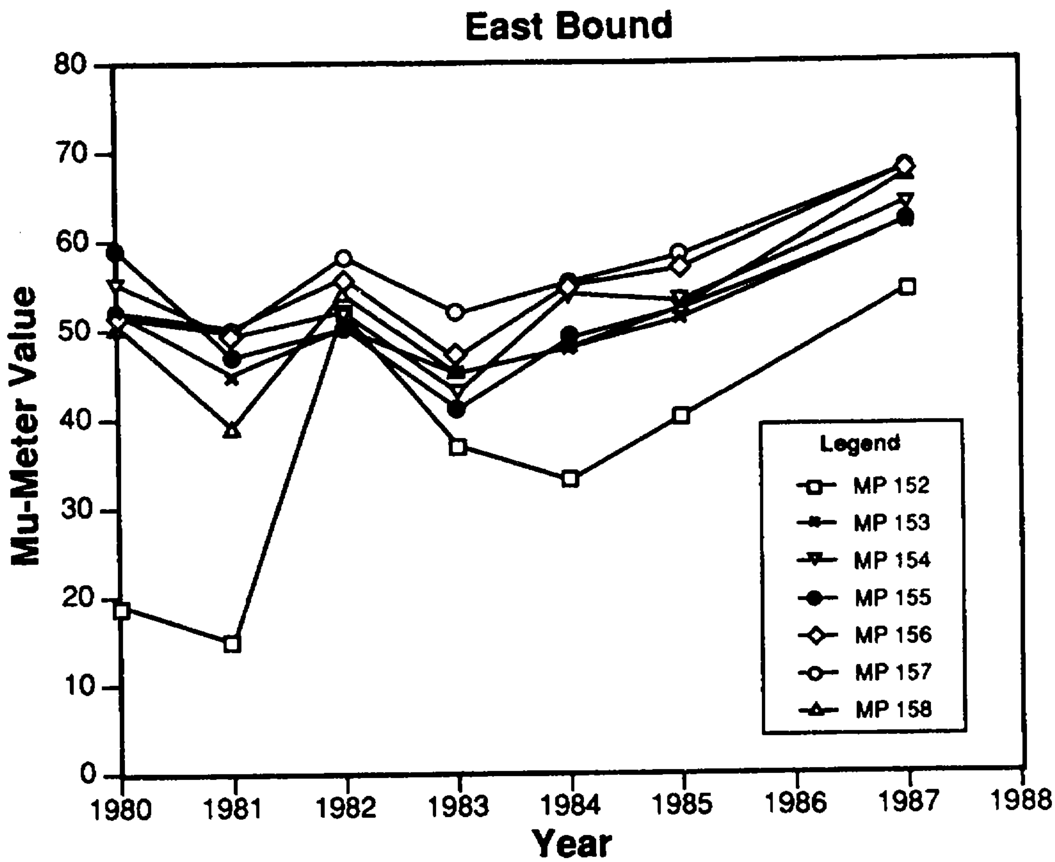
Figure 8- Mays Meter Roughness vs. Time

## TABLE 2 - ROUGHNESS REGRESSION RESULTS

| Direction | Milepost | Initial Roughness (1972 Data) (in/mi) | Roughness at Crack/Seat (in/mi) | Rate of Roughness Increase (in/mi/year) | $R^2$ |
|---|---|---|---|---|---|
| WB: | | | | | |
| | 153 | 95 | 203 | 6.7 | 0.84 |
| | 154 | 102 | 209 | 7.1 | 0.90 |
| | 155 | 116 | 285 | 12.2 | 0.90 |
| | 156 | 128 | 324 | 14.1 | 0.90 |
| | 157 | 91 | 253 | 10.4 | 0.83 |
| | Average | 106 | 255 | 10.1 | |
| EB: | | | | | |
| | 153 | 220 | 377 | 7.9 | 0.49 |
| | 154 | 174 | 294 | 7.5 | 0.71 |
| | 155 | 114 | 240 | 10.2 | 0.74 |
| | 156 | 134 | 369 | 16.0 | 0.88 |
| | 157 | 98 | 312 | 14.6 | 0.94 |
| | Average | 148 | 318 | 11.2 | |

## TABLE 3 - SKID REGRESSION RESULTS

| Direction | Milepost Location | Predicted Skid Value at Time of Crack/Seat | Rate of Skid Decrease (units/year) | $R^2$ |
|---|---|---|---|---|
| EB: | | | | |
| | 153 | 51 | -0.0285 | 0.0003 |
| | 154 | 53 | -0.11 | 0.002 |
| | 155 | 52 | -1.085 | 0.12 |
| | 156 | 57 | 0.890 | 0.18 |
| | 157 | 58 | 1.34 | 0.45 |
| | 158 | 52 | 0.8 | 0.08 |
| | Average | 54 | 0.30 | |
| WB: | | | | |
| | 153 | 62 | 1.46 | 0.76 |
| | 154 | 64 | 1.257 | 0.45 |
| | 155 | 62 | 1.51 | 0.45 |
| | 156 | 54 | 2.54 | 0.64 |
| | 157 | 45 | 1.34 | 0.56 |
| | 158 | 49 | 2.77 | 0.45 |
| | Average | 57 | 1.81 | |



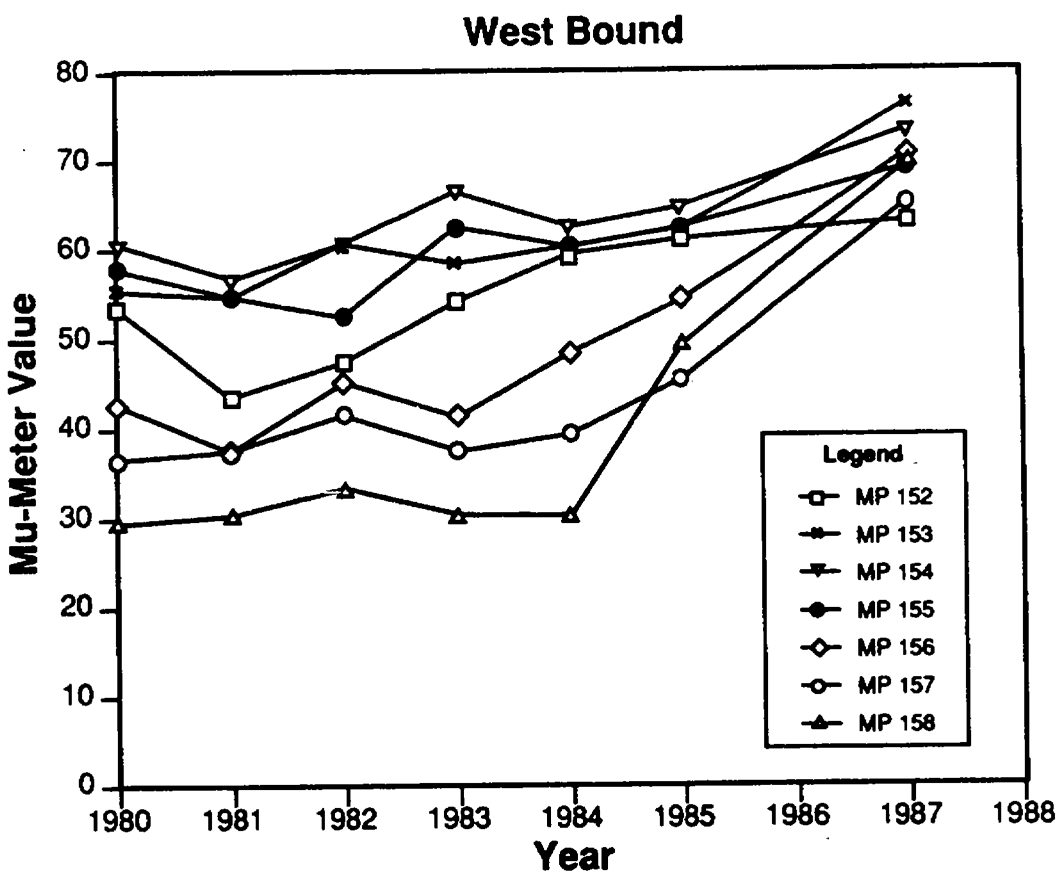




Figure 9 - Mu-meter Value vs. Time

## STRUCTURAL PAVEMENT PERFORMANCE

### Testing Performed and Purpose

The structural condition of the pavement was evaluated by pavement distress surveys, dynaflect testing, and extracting and testing PCCP cores. In addition to the historical crack, patch, and faulting data available in ADOT's PMS database, pavement distress surveys were conducted during the design phase.

### Laboratory Testing

Four 4 inch diameter cores were retrieved from the concrete pavement during the design phase. Compression tests with strain measurements were conducted on these cores to assess the structural integrity of the concrete. The results of this testing are shown in Table 4. The laboratory test results indicated that even though the compressive strength was 7000 to 9000 psi, the modulus was only two million psi. This rather low modulus was reported in the design summary as an indication of the poor concrete condition. Although the actual laboratory test conditions are not known at this time, the ACI relationship between compressive strength and modulus suggests that the modulus is of the order of five million psi.[4]

TABLE 4 - CONCRETE TEST RESULTS

| Location of Core | Compressive Strength (psi) | Laboratory Determined Modulus (psi) | Calculated Modulus ($57000\sqrt{f'c}$) (psi) |
|---|---|---|---|
| 153.02 EB | 7180 | $2.12\times10^6$ | $5.1\times10^6$ |
| 159.98 EB | 9019 | $2.41\times10^6$ | |
| 152.1 WB | 6614 | $1.54\times10^6$ | $5.0\times10^6$ |
| 159.0 WB | 8860 | $1.31\times10^6$ | |

### Fault Data

Figures 10 and 11 indicate the magnitude of faulting as a function of time for each roadway direction and milepost location. Data are not shown before 1981 because this information was not included in the PMS database. Faulting typically ranged between 1/10 inch and 3/10 inch throughout. A linear regression of traffic loading (expressed in million ESALs) and faulting (expressed in 1/10 inch) resulted in the following equations:

EB Faulting = 0.1038 + 3.46 * ESALs $\quad R^2 = 0.38$

WB Faulting = 0.821 + 0.801 * ESALs $\quad R^2 = 0.03$

The correlation for the EB direction is poor and the WB direction suggests no correlation. The slopes suggest that on the average, faulting in the WB direction was increasing less than the EB direction which is also reflected by the lower roughness in the WB direction.

Table 5 indicates the measured faulting at each milepost as reported in the PMS database in 1986. As noted, the faulting in the EB direction was approximately 3/10 inch while in the WB direction it was 2/10 inch.

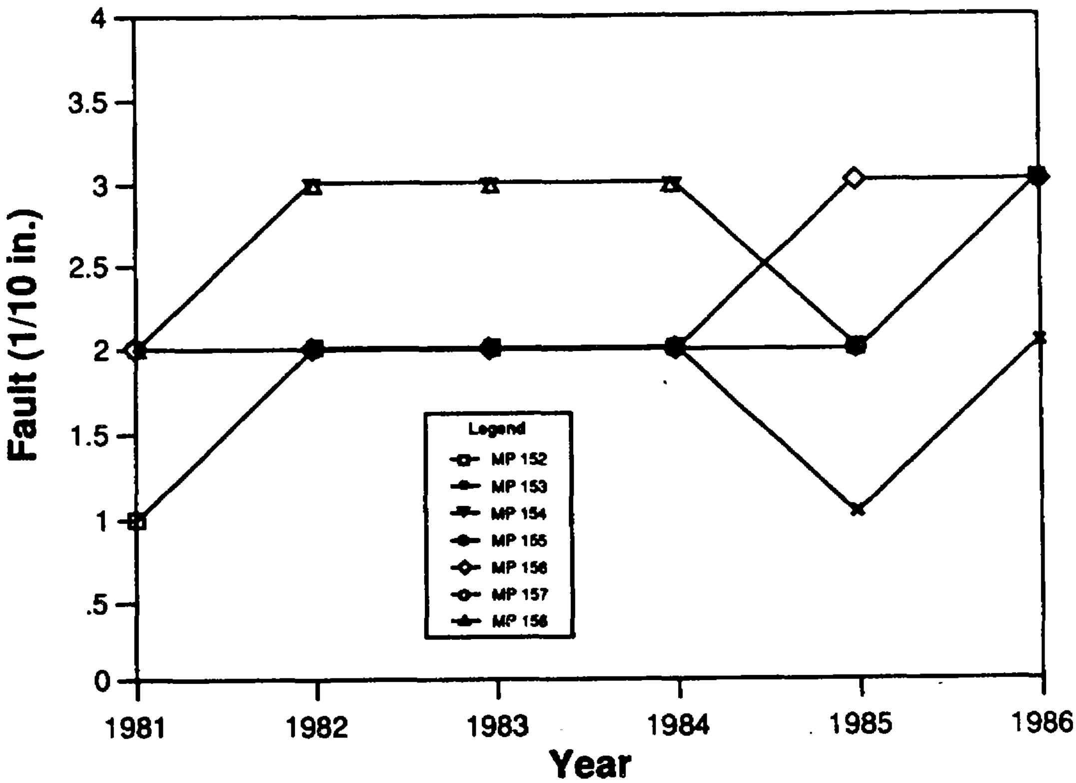



Figure 10- Fault vs. Time for the EB Direction

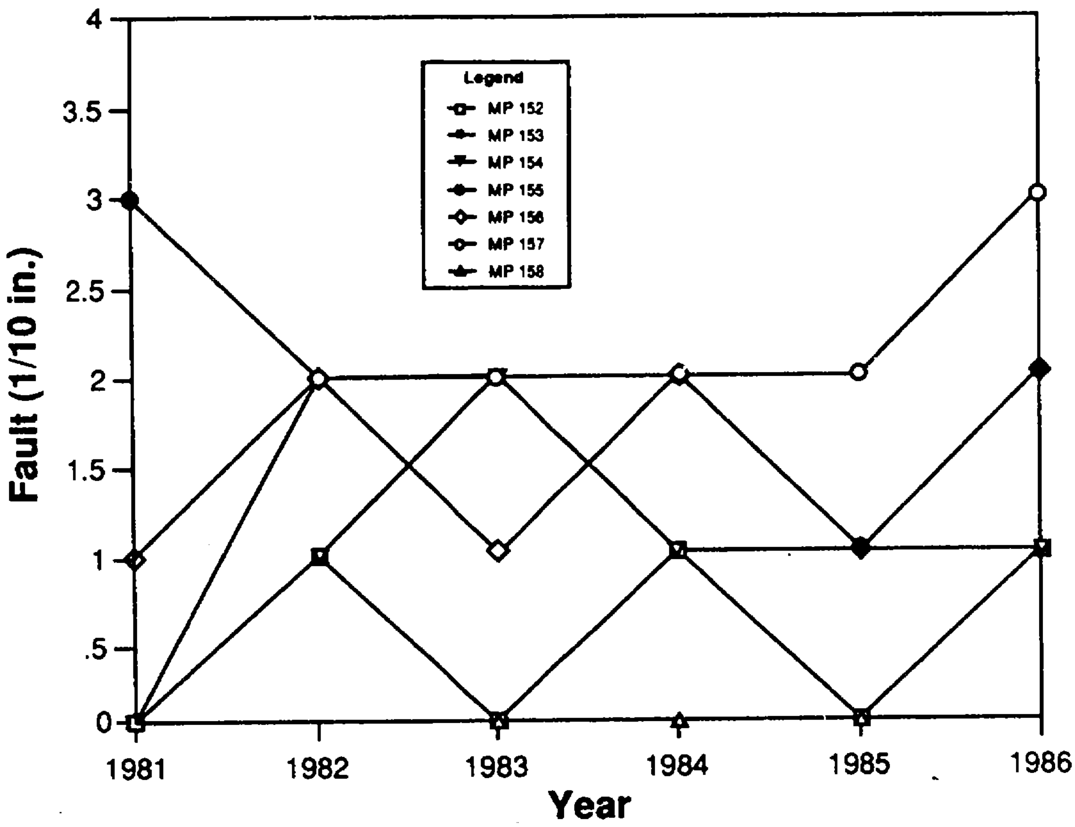



Figure 11- Fault vs. Time for the WB Direction

**Crack Data**

Figure 12 indicates the percent of cracking as a function of time for both directions of travel for each milepost location. No cracking is evident prior to 1981 because this data was not included in the PMS prior to this. It should be noted that at the time of construction (1986), cracking averaged 3.3% and 3.2% for the EB and WB roadways, respectively. Both roadways ranged between 0 and 5% cracking throughout the reporting period. At about 1984, most of the mile post locations showed a significant increase in crack development.

Table 5 shows the percent cracking at each milepost location for 1986 prior to the crack and seat project. It should be noted that the pavement design summary indicated that the project exhibited 40% cracking in the travel lane and 15% in the passing lane. The reason for the large disparity on the level of cracking reported in ADOT's PMS database and that reported in the design summary is not known. However, the concrete pavement exhibited "D" cracking on both roadways. It is therefore possible that the "D" cracking tended to inflate the designers estimate of cracking.

Only minimal patching existed on this project prior to the crack and seat construction. As evident in Table 5, patching was generally 1% or less throughout both roadways.

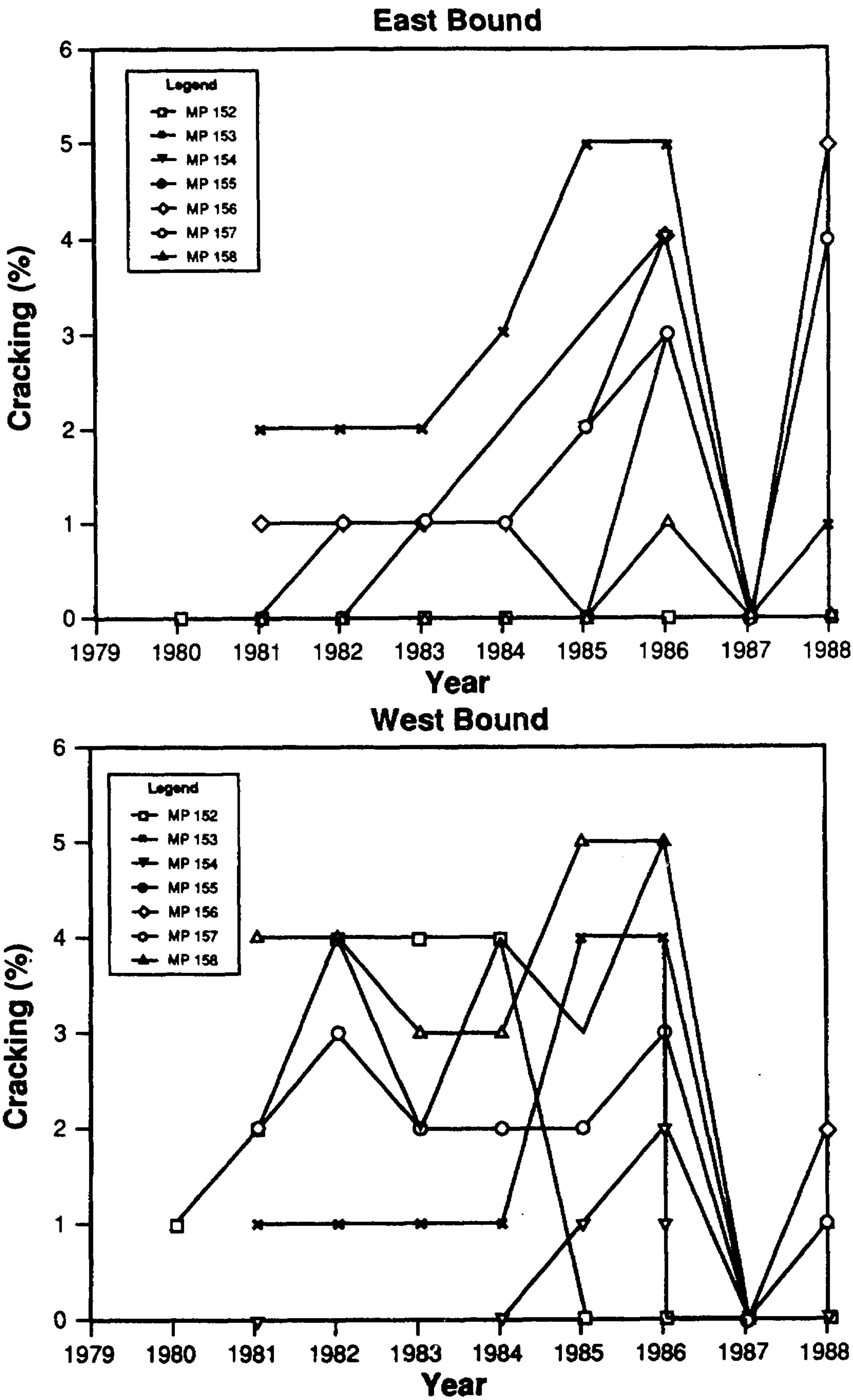



Figure 12- Cracking (%) vs. Time

TABLE 5 - STRUCTURAL DISTRESS DATA FOR 1986

| Location | Faulting (inches) | Cracking(%) | Patching(%) |
|---|---|---|---|
| 153.0 EB | 0.3 | 5.0 | 1.0 |
| 154.0 EB | 0.2 | 4.0 | 0.0 |
| 155.0 EB | 0.3 | 3.0 | 2.0 |
| 156.0 EB | 0.3 | 4.0 | 2.0 |
| 157.0 EB | 0.3 | 3.0 | 1.0 |
| 158.0 EB | 0.3 | 1.0 | 0.0 |
| 159.0 EB | 0.3 | 0.0 | 0.0 |
| Average | 0.29 | 3.3 | 1.0 |
| 153.0 WB | 0.01 | 4.0 | 0.0 |
| 154.0 WB | 0.02 | 2.0 | 0.0 |
| 155.0 WB | 0.01 | 0.0 | 0.0 |
| 156.0 WB | 0.02 | 5.0 | 1.0 |
| 157.0 WB | 0.02 | 3.0 | 1.0 |
| 158.0 WB | 0.03 | 5.0 | 0.0 |
| 159.0 WB | 0.01 | 0.0 | 0.0 |
| Average | 0.17 | 3.2 | 0.3 |

**Material Related Problems**

As previously discussed, both roadways exhibited considerable "D" cracking prior to the crack and seat project. Since ADOT's PMS does not report "D" cracking, no records are available on the development of this distress with time.

**Deflection Testing**

Dynaflect deflection testing was performed in April 1985 to evaluate the structural capacity and load transfer efficiency of the existing pavement. The peak to peak test load was approximately 1100 pounds at a frequency of 8 MHz. A total of 429 deflection tests were conducted between MP 152.5 and MP 158. Tests were conducted at various locations at half mile intervals in each lane in both directions. The deflection test results on the travel lanes at half mile intervals are shown in Appendix A. A statistical analysis comparing the deflection readings of the number 1 and 5 sensors was performed. At a significance level of 5% there was no difference in the two sensor deflections for either roadway. This suggests that structural behavior of the PCCP was essentially similar for both directions. A summary of the dynaflect results are shown in Table 6.

TABLE 6 - DYNAFLECT DEFLECTION TEST RESULTS

| | EB Roadway | | | | | WB Roadway | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Statistic | 1* | 2* | 3* | 4* | 5* | 1* | 2* | 3* | 4* | 5* |
| $\bar{X}$ | .70 | .65 | .50 | .36 | .26 | .89 | .70 | .62 | .44 | .32 |
| Std. Dev | .297 | .255 | .199 | .168 | .118 | .456 | .252 | .197 | .134 | .110 |
| C.V. | 42% | 39% | 40% | 47% | 45% | 51% | 36% | 32% | 31% | 34% |
| Range | .43-1.57 | .4-1.37 | .33-1.07 | .06-.76 | .04-.49 | .52-1.91 | .5-1.1 | .48-.92 | .33-.63 | .24-.48 |

* Sensor number.
Note: All deflection readings are in mils

The average coefficient of variation for deflections was 43% and 37% for the EB and WB roadways, respectively.

**Load Transfer Efficiencies**

Deflection testing was also conducted to determine the load transfer effectiveness of both cracked and uncracked joints. The test locations and the results are shown in Appendix A. Tests were conducted in both lanes of both roadways. The load transfer efficiencies of the deteriorated and undeteriorated joints in the travel lanes were compared statistically. At a significance level of 5%, there was no difference between the load transfer efficiencies of deteriorated and undeteriorated joints for any direction. Statistics regarding load transfer efficiencies of the joints for both lanes of each direction of roadway are shown in Table 7.

The results in Table 7 show that the coefficients of variation of the load transfer efficiencies of the joints in the EB direction for the travel lane are very high for deteriorated and undeteriorated joints. The average load transfer efficiencies for the travel for the EB direction for both deteriorated and undeteriorated joints were much less than those for the WB direction. The joints in the EB direction appeared to have serious load transfer problems which is also evident by the higher average faulting (3/10 inches) in that direction. The pavement in the EB direction was built one year later than pavement in the WB direction and also had a different aggregate source. Assuming that the environment and loading conditions were similar for those two directions of roadway, construction and material related problems can be attributed to the varying structural performance of the pavements in two directions.

TABLE 7 - DYNAFLECT LOAD TRANSFER EFFICIENCIES

| | | EB Roadway | | WB Roadway | |
|---|---|---|---|---|---|
| Joint Condition | Statistic | Lane 1* | Lane 2** | Lane 1* | Lane 2** |
| Undeteriorated | $\bar{X}$ | 0.98 | 0.65 | 0.96 | 0.80 |
| | Std. Dev. | 0.018 | 0.42 | 0.03 | 0.22 |
| | C.V. | 1.9 | 68.2 | 3.55 | 28.6 |
| | Range | 0.95-1.0 | 0.05-1.00 | .88-0.99 | 0.45-0.98 |
| Deteriorated | $\bar{X}$ | 0.97 | 0.55 | 0.77 | 0.67 |
| | Std. Dev. | 0.029 | 0.36 | 0.21 | 0.24 |
| | C.V. | 3.16 | 68.8 | 28.7 | 37.5 |
| | Range | 0.93-1.0 | 0.04-1.0 | 0.48-0.96 | 0.33-0.99 |

*Passing Lane
**Travel Lane

## MAINTENANCE COSTS

### Description of Activities

Significant maintenance activities were performed on this project just prior to the crack and seat project. Appendix B shows the maintenance activities and associated costs for this project during the period 1980-1986.

Figure 13 represents the reported maintenance costs associated with I-40 within the construction project limits for the period 1980-1988. As noted, very high maintenance costs are evident during 1985 and 1986 just before the crack and seat construction. High maintenance costs were also incurred during the year immediately following construction (1987). This was due to the application of a flush coat to the newly constructed overlay to prevent stripping and ravelling approximately six months after the construction.

Maintenance prior to the construction project typically consisted of leveling with premix, filling cracks, and flush coats on the AC shoulders.

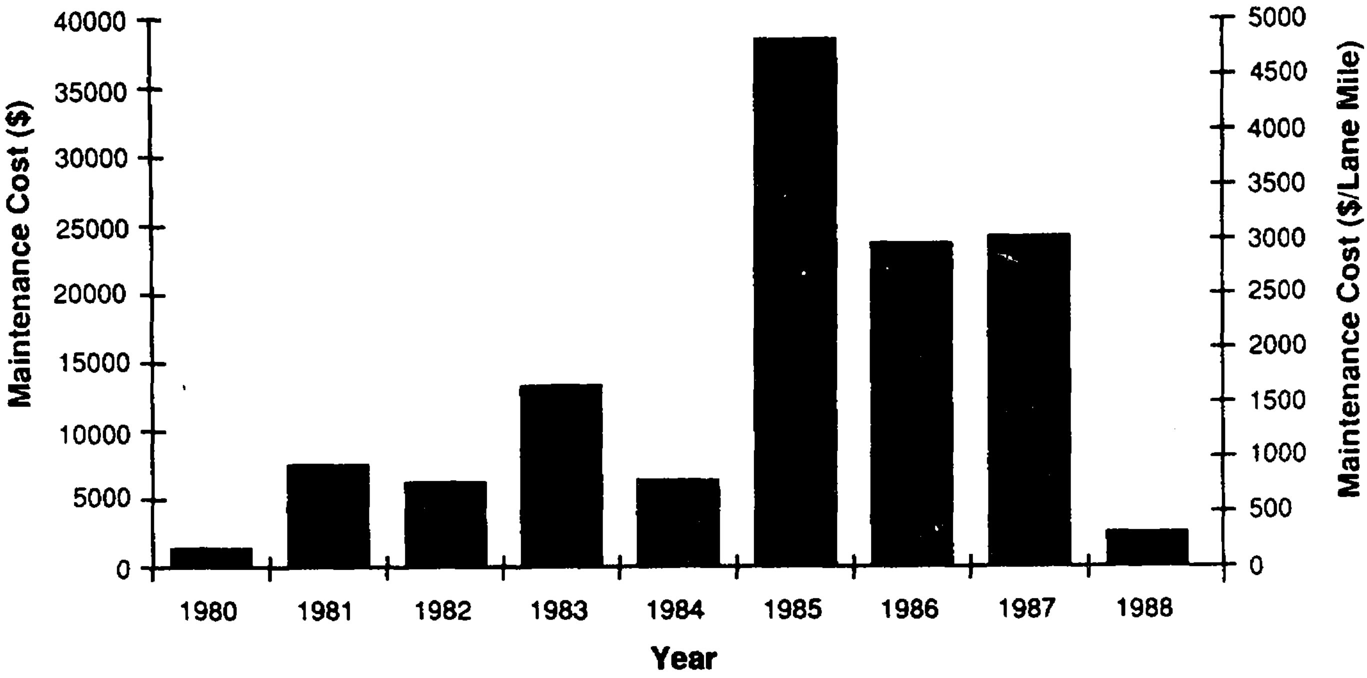



Figure 13- Roadway Maintenance Cost for the Crack and Seat Project for the Years 1980-1988.

## DESIGN EVALUATION

### ADOT Design Procedure

Since ADOT does not have explicit design procedures for overlaying concrete pavements, engineering judgement was utilized to develop the pavement design. Field testing consisted of conducting dynaflect testing, retrieving cores, and obtaining subgrade samples. Once the material properties were assessed, the design practice of other agencies was reviewed to assist in the decision process. Three alternatives were considered; (1) crack and seat with 4" overlay, (2) seal joints and cracks, place Stress Absorbing Membrane Interlayer (SAMI) and 3" overlay, and (3) pulverize existing concrete and place 8 1/2" AC surfacing.

The crack and seat with 4" overlay resulted in approximately the same estimated construction costs as Alternative 2. Alternative 3 was considerably more expensive. Since the design recommendations were to provide additional structure to the section and reduce roughness, the crack and seat alternative was selected.

## CONSTRUCTION PROBLEMS

During the paving of the bottom 2 inch lift of the HMAC overlay, bumps were observed directly above the transverse joints in the PCCP slab. The areas with severe bumps (height equal to or greater than 1/8 inch) were milled off. The bumps were attributed to the rubber sealants in the joints. It should be noted that the bumps were not seen in the top 2 inch lift.

The AC density attainment was difficult at times. The bottom 2 inch lift of HMAC produced a total of 19 failing lots, while the top 2 inch lift produced 12 failing lots.

For the test sections between MP 152.1 and MP 152.3 WB, lots 35 and 39 showed 55% and 86% compliance, respectively, for the bottom 2 inch lift, while for the top 2 inch lift, lots 62 and 56 showed a 17% and 91% compliance, respectively.[1]

Problems were also experienced with the trench design. During construction lane closures, traffic tended to wander over the trench drain location overstressing the AC surfacing. The minimal 2 inch AC surface over the trench location was inadequate to sustain the traffic loadings.

## FIELD INVESTIGATION SINCE CONSTRUCTION

### Pavement Distress Surveys

Since the completion of the project in November of 1986, there have been three visual distress surveys. Surveys were conducted by the ATRC at 9 months, 17 months, and 26 months. The first evidence of pavement distress was noted after only 3 months when four cracks were noted between MP 155-156 on the EB roadway. At the 9 month survey only two cracks were noted in the WB roadway. These were evident between MP 157 and 158. In the eastbound roadway, several cracks were reported between MP 153 and 154 and MP 155 and 156. All the cracking reported on both roadways was observed in the travel lane and on sections located on an uphill grade. No cracking was observed in the passing lanes or on level grades. Occasional longitudinal cracking was observed over the trench drain between MP 153 and MP 154.

By the 17 and 26 month surveys, considerable reflection cracking had occurred on this project. Figure 14 indicates the number of transverse cracks observed in the travel lane during these surveys. The eastbound roadway exhibited considerably more cracking at the 17 and 26 month surveys. The eastbound roadway also experienced the largest increase in the amount of cracking between the 17 and 26 month surveys. Both roadways exhibited increased cracking when proceeding from the west end of the project in an easterly direction until milepost 156-157 where the cracking occurrence diminished.

The 17 and 26 month surveys consisted of a visual reconnaissance of the roadways. The cracking was determined by driving along the shoulder at 10 to 15 MPH and counting any transverse crack which existed in the travel lane.

No discernible rutting was evident during the "windshield" surveys. However, slight raveling was evident throughout, particularly at the longitudinal construction joints. Although, no measurements were taken, there was no evidence of vertical displacement across the transverse cracks in the overlay.

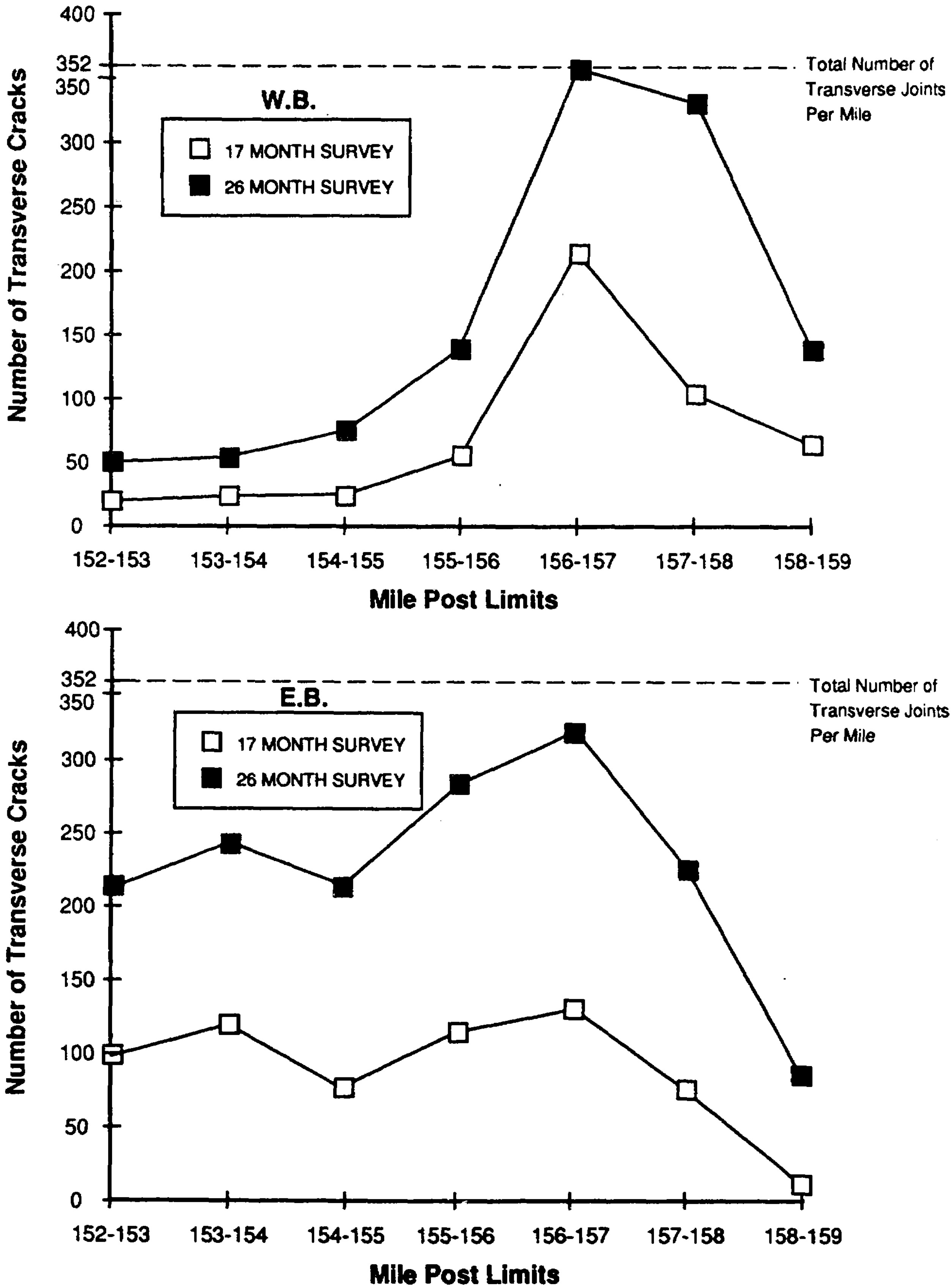



Figure 14- Transverse Cracking on the Project since Crack and Seat Construction

## NON DESTRUCTIVE TESTING

### Purpose and Background

Non destructive testing (NDT) was conducted at 9, 17 and 33 months after construction and consisted of deflection testing using a falling weight deflectometer (FWD). The FWD testing was performed with a Dynatest model 8002 with sensor locations of 0, 12, 24, 36, 48, 60, and 72 inches from the center load at load levels of 6,000, 9,000, and 12,000 pounds. The testing was performed at 39 locations in 1987, 43 locations in 1988, and 17 locations in 1989. Figure 15 indicates the locations where the FWD testing was conducted and the locations of the two test sections and the 3 ft by 3 ft "control" section. The FWD test location mileposts are shown in Appendix C.

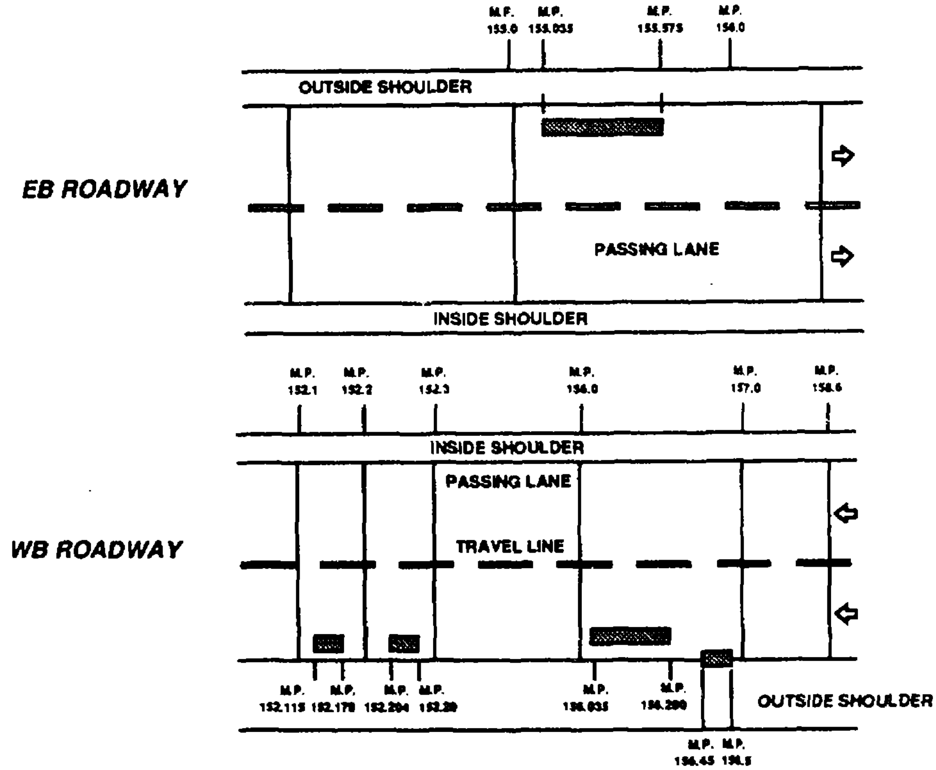



Figure 15- NDT Test Locations (1987, 1988 and 1989 Testing).

The FWD testing was performed to establish the effectiveness of the crack and seat operation and to determine how the effective modulus of the concrete pavement changed with time. Since the Michigan Whiphammer imparted so little fracture energy into the pavement during construction, it was assumed that the effective modulus would decrease with time as the effects of thermal changes and freeze thaw acted to reduce the aggregate interlock at the fracture locations.

### Analysis Methodology

The FWD deflection data were analyzed using the BKCHEVM elastic layer computer program to backcalculate the layer moduli.[5] Two test sections (2 ft by 2 ft and 6 ft by 4 ft crack spacing) and one "control" section (3 ft by 3 ft crack spacing) were selected for back calculation of layer moduli. The pavement was assumed to be a 4-layer flexible pavement system. Layer 1 was the AC overlay, layer 2 was the cracked and seated PCCP, layer 3 was the Cement Treated Base(CTB), and layer 4 was the subgrade which was assumed to be semi-infinite. The deflection values corresponding to the load level closest to 9000 pounds were used for backcalculation of layer moduli.

### Backcalculation Results

The results of the backcalculation analysis are summarized in Table 8. The individual test location results are included in Appendix D. As shown in Table 8, very high concrete pavement moduli were obtained for the 1987, 1988, and 1989 analysis. The average JPCP moduli typically ranged between 3.3 and 7 million psi. Individual test results ranged from a low of 140,000 psi to a high of 11.7 million psi. The large standard deviation and coefficient of variation existed for the 1987, 1988, and 1989 data. The average moduli for test sections 1, 2, and the "control section" suggest a fluctuation of cracked PCCP layer moduli with time. No definite trend was evident to support a decrease of cracked PCCP layer modulus with time.

TABLE 8 - BACKCALCULATED LAYER MODULI SUMMARY STATISTICS

| Layer | Year | Test Section 1 MP 152.2 - 152.3 WB 2' x 2' | | | Test Section 2 MP 152.2 - 152.1 WB 6' x 4' | | | Control Section MP 156.035 - 156.290 WB 3' x 3' | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | Average Modulus (psi) | Std. Deviation (psi) | Coefficient of Variation | Average Modulus (psi) | Std. Deviation (psi) | Coefficient of Variation | Average Modulus (psi) | Std. Deviation (psi) | Coefficient of Variation |
| Asphalt Concrete (AC) | 1987 | 189,702 | 103,491 | 0.545 | 203,237 | 43,854 | 0.215 | 208,254 | 45,602 | 0.22 |
| | 1988 | 185,640 | 39,730 | 0.214 | 241,225 | 13,054 | 0.054 | 401,610 | 332,017 | 0.83 |
| | 1989 | 664,745 | 341,633 | 0.514 | 683,080 | 373,364 | 0.546 | 1,019,351 | 309,652 | 0.304 |
| Broken Portland Cement Concrete (PCC) | 1987 | 6,491,073 | 3,315,974 | 0.51 | 6,671,760 | 3,484,314 | 0.52 | 5,771,662 | 3,499,824 | 0.606 |
| | 1988 | 3,909,532 | 2,196,941 | 0.562 | 4,875,500 | 3,598,308 | 0.738 | 6,223,633 | 3,584,929 | 0.576 |
| | 1989 | 8,083,204 | 4,151,820 | 0.513 | 5,676,111 | 5,553,163 | 0.978 | 3,343,581 | 3,106,757 | 0.929 |
| Cemented Treated Base (CTB) | 1989 | 43,333 | 15,877 | 0.366 | 58,125 | 11,250 | 0.193 | 81,633 | 62,617 | 0.767 |
| | 1988 | 44,375 | 7,181 | 0.162 | 42,500 | 8,660 | 0.204 | 78,007 | 26,994 | 0.346 |
| | 1987 | 69,375 | 11,250 | 0.162 | 56,250 | 13,346 | 0.237 | 122,822 | 167,300 | 1.36 |
| Subgrade | 1989 | 23,349 | 3,204 | 0.137 | 26,856 | 6,161 | 0.229 | 34,396 | 6,592 | 0.27 |
| | 1988 | 23,588 | 3,437 | 0.146 | 28,547 | 3,117 | 0.117 | 16,180 | 5,517 | 0.303 |
| | 1987 | 20,320 | 1,647 | 0.081 | 22,691 | 4,115 | 0.181 | 27,783 | 2,558 | 0.092 |

Backcalculations were performed on FWD test results from four locations within section 1 and four locations within section 2. Ten test locations were analyzed for the control section. If the backcalculation results are valid, they suggest that little reduction in effective modulus has resulted from the crack and seat process. The backcalculated moduli are comparable to the results calculated from the compressive test results (5 million psi) of the concrete cores obtained during the design phase. Additionally, higher 1988 average moduli in the control section corresponds to a higher incidence of reflective cracking observed between MP 156 and 157.

It should be noted that the existence of a rigid-layer (cracked and seated PCCP) between two flexible layers violates the basic configuration of a layered elastic system where moduli are assumed to decrease with depth. Similarly, the effect of a rigid bottom was not accounted for in these analysis as a semi-infinite subgrade was assumed. Therefore, the backcalculation analysis is of questionable validity.

### Determination of Structural Layer Coefficients

The results of the backcalculation analysis were used to calculate the structural layer coefficients using both the 1986 AASHTO Guide procedures and the modified NAPA procedures.[a,6,7] The AASHTO procedures are explained in the guide. The AASHTO procedure is valid for cracked and seated concrete moduli up to 950,000 psi. Since most of the backcalculated PCCP layer moduli values of this project were far above 950,000 psi, the layer coefficients corresponding to the moduli values greater than 950,000 psi have been taken as 0.44, the maximum layer coefficient for cracked and seated concrete in the AASHTO Guide.

The NAPA equation for computing structural layer coefficient of cracked and seated concrete is as follows:

$$a_{cs} = a_2(E_{cs}/E_2)^{1/3}$$

where: $a_{cs}$ = Structural layer coefficient of cracked and seated concrete

$E_{cs}$ = Effective modulus of cracked and seated layer (psi)

$a_2$ = Layer coefficient for the base (CTB)

$E_2$ = Layer modulus of the base (CTB)

The cement treated base layer coefficient and modulus were held fixed throughout the analysis for a valid comparison of change in effective modulus over time and also between the crack spacings.

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a Backcalculation for determination of effective modulus of cracked and seated concrete was done by BKCHEVM. Only the equation for computing structural layer coefficient of cracked and seated concrete in NAPA procedure has been used. The cement treated base layer coefficient and modulus were assumed to be 0.12 and 500,000 psi respectively.

TABLE 9 - VARIATION OF PCCP STRUCTURAL LAYER COEFFICIENTS WITH CRACK SPACING AND TIME

| Year | Crack Spacing | PCCP Structural Layer Coefficient (AASHTO) | PCCP Structural Layer Coefficient (NAPA) |
|---|---|---|---|
| 1987 | 2' x 2' | 0.44 | 0.27 |
| | 3' x 3' | 0.44 | 0.255 |
| | 6' x 4' | 0.42 | 0.24 |
| 1988 | 2' x 2' | 0.44 | 0.23 |
| | 3' x 3' | 0.44 | 0.26 |
| | 6' x 4' | 0.44 | 0.24 |
| 1989 | 2' x 2' | 0.44 | 0.29 |
| | 3' x 3' | 0.44 | 0.206 |
| | 6' x 4' | 0.39 | 0.229 |

The calculated structural layer coefficients for both the AASHTO procedures and the modified NAPA procedures are shown in Table 9. These results indicate the variation in layer coefficients with both time and spacing. These variations are shown graphically in Figures 16 and 17. The modified NAPA procedures tended to give the highest coefficient for the 2 ft spacing for 1987 and 1989 data.

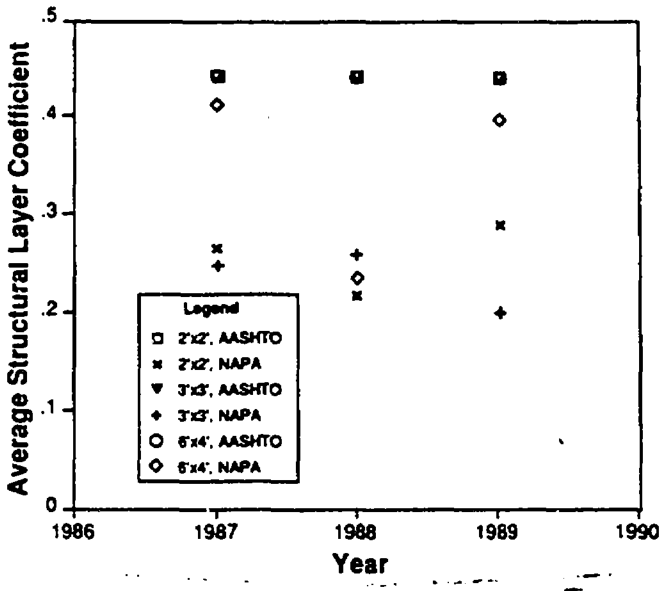


Figure 16- Structural Layer Coefficients vs. Time

Figure 17 indicates that the modified NAPA procedures resulted in decreased coefficients with time for only the 6 ft by 4 ft crack pattern.

The NAPA coefficients were generally less than the magnitude of the AASHTO coefficients. In most of the cases, AASHTO coefficients have been taken as the highest coefficient tabulated in the Guide. It should be noted that calculation of structural coefficient of crack and seated concrete in the NAPA procedure is very sensitive to the structural coefficient of the base layer. Since the base layer coefficient and modulus have been selected arbitrarily, no comparison between AASHTO layer coefficients and NAPA layer coefficients are valid. Also, by comparing the structural layer coefficients no reasonable conclusion can be drawn about the performance of one crack spacing over another.

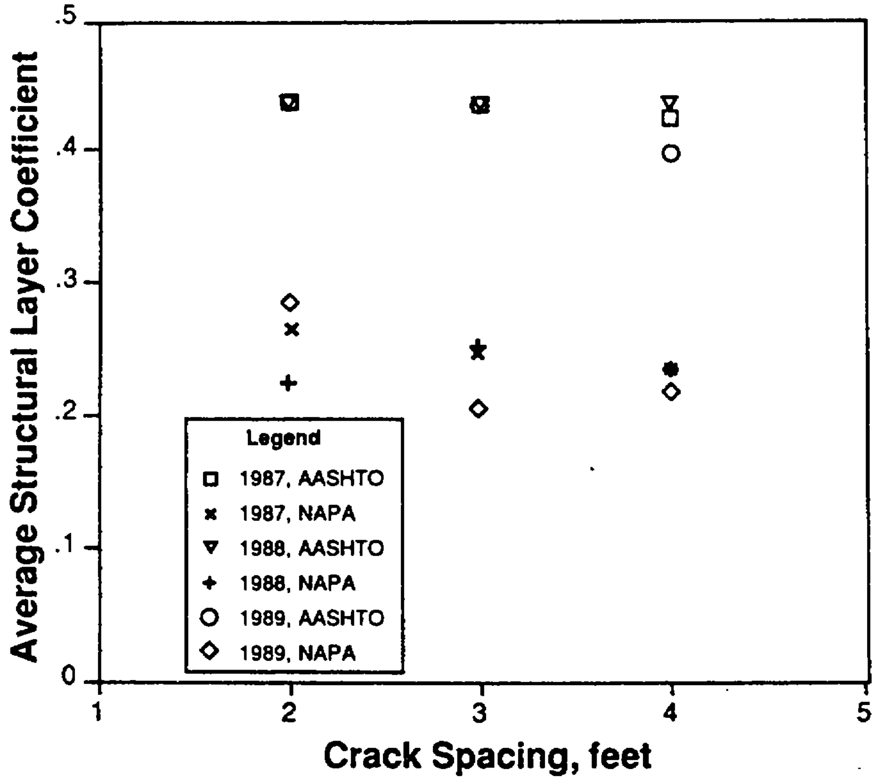

Figure 17- Structural Layer Coefficients vs. Crack Spacing

## PAVEMENT PERFORMANCE SINCE OVERLAY

### Functional Performance

#### Roughness

The crack and seat overlay project resulted in an average decrease in roughness of 254 inches/mile for the eastbound roadway and 165 inches/mile for the westbound roadway. The current roughness values are probably lower than that attained by the concrete pavement when it was first constructed. The recent construction project was very successful in reducing roughness.

### Skid

Only one year of Mu-meter data is available since the construction of this project. With the systematic errors attendant to the Mu-meter operation comparisons with past performance are difficult. However, the PMS data indicates an increase of 11 units and 13 units for the eastbound and westbound directions, respectively.

### Maintenance Costs

The historical maintenance costs previously shown in Figure 13 indicate a significant maintenance cost in 1987 and a sharp reduction in 1988 expenditures. Although the 1987 expenditures resulted from the need to flush the surface course to prevent further raveling, the expenditures in 1989 are also anticipated to be high. Extensive crack sealing has occurred during the current year. The authors do not believe the recent construction project has been effective in reducing maintenance expenditures.

### Structural Performance

The addition of a 4 inch AC surfacing has certainly increased the structural capacity of the pavement. However, the extent and severity of transverse cracking suggests that the crack and seat and four inch overlay were not successful in mitigating reflective cracking. At this time it is not possible to determine whether the crack and seat operation was ineffective, the asphalt concrete mix design was inappropriate, or a combination of both. The rapidity with which the concrete joints reflected through the four inch overlay suggests the need to design "special" AC mixtures for overlaying concrete pavements. The use of standard mixes designed without special consideration for prevention of reflective cracking appears inappropriate.

## CONCLUSIONS AND RECOMMENDATIONS

### Conclusions

The crack and seat with four inch overlay strategy was very successful in reducing roughness. However, it was ineffective at preventing reflective cracking. It is very questionable as to whether it was even effective at mitigating the reflective cracking problem.

The roadway between MP 156 and MP 158 has exhibited significantly more transverse cracking than the other sections for both roadway directions.

Large variations in the effective modulus of the cracked concrete pavement were obtained from the backcalculation procedures. Unreasonably high PCCP moduli values appear to have been computed through this process. PCCP moduli for different crack spacing tended to fluctuate over time. No consistent trends were evident.

AASHTO procedure for computing structural layer coefficient was not applicable in most of the data for this project because of unusually high cracked and seated concrete moduli. AASHTO procedure has tabulated values of structural layer coefficients for cracked and seated concrete moduli up to 950,000 psi.

The structural layer coefficients predicted by the modified NAPA procedures appear to decrease with time for 6 ft by 4 ft spacing. Layer coefficient analysis did not indicate any advantage to any of the crack spacings.

**Recommendations**

Future crack and seat projects should include a control section which is not cracked and seated. This will provide a basis for comparing whether the crack and seat procedures are effective.

Asphalt concrete overlays placed upon concrete pavements should be designed to mitigate reflective cracking. Standard mix designs should not be used.

Future research on this project should evaluate the asphalt concrete mix properties and performance, the significant difference in transverse crack occurrence throughout the project, and any longitudinal cracking occurring over the trench drain.

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3 "Climates of the States," Volume 1: Alabama - New Mexico, 3rd Edition, pp. 40-58, Published by Gale Research Company, Detroit, Michigan, 1985.

4 "ACI Building Code," Published by the American Concrete Institute (ACI), Detroit, Michigan, 1977.

5 BKCHEVM, Modified Microcomputer version of BKCHEV Backcalculation Program. Modified in the project, "Rational Characterization of Pavement Structures Using Deflection Analysis," Project Number HPR-PL-1(31)-254, Arizona Department of Transportation, December, 1988.

6 "AASHTO Guide for Design of Pavement Structures," Published by American Association of State Highway and Transportation Officials (AASHTO), Washington D.C., 1986.

7 "Structural Evaluation of Cracked and Seated PCC Pavements for Overlaying with Hot Mix Asphalt," National Asphalt Pavement Association (NAPA) Information Series 100, 1987.

## APPENDIX A

TABLE A-1 - DYNAFLECT DEFLECTION TEST RESULTS FOR THE TRAVEL LANE IN THE EB DIRECTION (MP 152.3 - 158.6)

| MP Location | Dynaflect Readings (mils) #1* | #2* | #3* | #4* | #5* |
|---|---|---|---|---|---|
| 152.50 EB | 0.49 | 0.46 | 0.34 | 0.21 | 0.14 |
| 153.00 EB | 0.76 | 0.69 | 0.55 | 0.39 | 0.28 |
| 153.50 EB | 0.43 | 0.40 | 0.33 | 0.27 | 0.21 |
| 154.00 EB | 0.75 | 0.85 | 0.33 | 0.06 | 0.04 |
| 154.50 EB | 0.58 | 0.54 | 0.41 | 0.27 | 0.19 |
| 155.00 EB | 0.83 | 0.68 | 0.48 | 0.33 | 0.24 |
| 155.50 EB | 1.57 | 1.37 | 1.07 | 0.76 | 0.49 |
| 156.00 EB | 0.54 | 0.51 | 0.45 | 0.38 | 0.31 |
| 156.50 EB | 0.63 | 0.62 | 0.56 | 0.46 | 0.38 |
| 157.00 EB | 0.66 | 0.62 | 0.54 | 0.44 | 0.36 |
| 157.50 EB | 0.56 | 0.51 | 0.42 | 0.32 | 0.23 |
| 158.00 EB | 0.61 | 0.59 | 0.47 | 0.37 | 0.28 |
| $\bar{X}$ = | 0.70 | 0.65 | 0.496 | 0.355 | 0.263 |
| Std. Dev. = | 0.297 | 0.255 | 0.199 | 0.168 | 0.118 |
| C.V. = | 42% | 39% | 40% | 47% | 45% |

* sensor number

TABLE A-2 - DYNAFLECT DEFLECTION TEST RESULTS FOR THE TRAVEL LANE IN THE WB DIRECTION (MP 152.3 - 158.6)

| MP Location | Dynaflect Readings (mils) #1* | #2* | #3* | #4* | #5* |
|---|---|---|---|---|---|
| 152.50 WB | 0.57 | 0.54 | 0.49 | 0.36 | 0.27 |
| 153.00 WB | 0.97 | 0.92 | 0.87 | 0.63 | 0.48 |
| 153.50 WB | 0.52 | 0.51 | 0.48 | 0.37 | 0.28 |
| 154.00 WB | 1.23 | 1.10 | 0.92 | 0.60 | 0.42 |
| 154.50 WB | 0.83 | 0.81 | 0.79 | 0.60 | 0.49 |
| 155.00 WB | 0.55 | 0.52 | 0.51 | 0.39 | 0.31 |
| 155.50 WB | 1.23 | 1.06 | 0.75 | 0.42 | 0.24 |
| 156.00 WB | 1.91 | 0.50 | 0.45 | 0.33 | 0.26 |
| 157.50 WB | 0.64 | 0.64 | 0.61 | 0.44 | 0.34 |
| 158.00 WB | 0.48 | 0.41 | 0.35 | 0.22 | 0.15 |
| $\bar{X}$ = | 0.893 | 0.701 | 0.622 | 0.436 | 0.324 |
| Std. Dev. = | 0.456 | 0.252 | 0.197 | 0.134 | 0.110 |
| C.V. = | 51% | 36% | 32% | 31% | 34% |

* sensor number

TABLE A-3A -AVERAGE LOAD TRANSFER EFFICIENCIES
FOR EAST BOUND ROADWAY

| | Undeteriorated Joint | | Deteriorated Joint | |
|---|---|---|---|---|
| Milepost | Lane 1 | Lane 2 | Lane 1 | Lane 2 |
| 152.50 | 0.99 | - | - | 0.90 |
| 153.00 | 1.0 | - | - | 0.95 |
| 153.50 | 0.97 | - | - | 0.33 |
| 154.00 | 0.97 | 0.05 | 0.98 | 0.04 |
| 154.50 | 0.95 | - | - | 0.10 |
| 155.00 | 0.96 | 0.45 | - | 0.48 |
| 155.50 | 0.99 | 0.08 | 0.97 | 0.31 |
| 156.00 | - | - | 0.93 | 0.86 |
| 156.50 | 0.98 | 0.89 | - | 1.0 |
| 157.00 | 1.0 | 1.0 | 1.0 | - |
| 157.50 | 1.0 | 1.0 | - | - |
| 158.00 | 0.99 | 1.0 | - | - |

TABLE A-3B - AVERAGE LOAD TRANSFER EFFICIENCIES
FOR WEST BOUND ROADWAY

| | Undeteriorated Joint | | Deteriorated Joint | |
|---|---|---|---|---|
| Milepost | Lane 1 | Lane 2 | Lane 1 | Lane 2 |
| 152.50 | 0.97 | 0.98 | - | - |
| 153.00 | 0.99 | 0.98 | - | 0.99 |
| 153.50 | 0.98 | 0.98 | - | - |
| 154.00 | 0.97 | 0.94 | - | 0.91 |
| 154.50 | 0.98 | 0.92 | - | - |
| 155.00 | 0.96 | 0.72 | - | - |
| 155.50 | 0.96 | - | - | 0.66 |
| 156.00 | - | - | 0.96 | 0.61 |
| 156.50 | 0.98 | - | 0.72 | - |
| 157.00 | 0.88 | - | 0.48 | - |
| 157.50 | 0.98 | 0.47 | - | 0.33 |
| 158.00 | 0.92 | 0.45 | 0.93 | 0.49 |

TABLE A-4 - PAIRED t-TEST RESULTS FOR TESTING THE DIFFERENCE IN #1 AND #5 SENSOR READINGS IN EACH DIRECTION

| Variable | Mean Difference Between EB & WB Direction | t | $t_{\alpha/2,dof}$ | Conclusion |
|---|---|---|---|---|
| #1 Sensor Reading | 1.81 | 1.18 | 2.26 | Not Significant* |
| #5 Sensor Reading | 0.083 | 1.38 | 2.26 | Not Significant* |

* at 5% Level of Significance.

TABLE A-5 - PAIRED t-TEST RESULTS FOR TESTING THE DIFFERENCE IN LOAD TRANSFER EFFICIENCIES OF UNDETERIORATED AND DETERIORATED JOINTS FOR THE TRAVEL LANES

| Lane | Mean Difference | t | d.o.f. | $t_{\alpha/2,dof}$ | Conclusion |
|---|---|---|---|---|---|
| E2 | 0.09 | 1.76 | 3 | 3.182 | Not Significant* |
| W2 | 0.0275 | 0.742 | 4 | 2.776 | Not Significant* |

* at 5% Level of Significance.

APPENDIX B

TABLE B-1 - MAINTENANCE ACTIVITIES PRIOR TO CRACK AND SEAT

| Year | Activity Number | Description of Activity | Associated Cost ($) |
|---|---|---|---|
| 1980 | 103 | Fill cracks | 765 |
| | 111 | Temp. Hand Patch | 299 |
| 1981 | 102 | Level with Premix | 3,400 |
| | 103 | Fill Cracks | 3,677 |
| | 119 | Pavement Surface Maint. | 525 |
| 1982 | 102 | Level with Premix | 2,031 |
| | 111 | Temp. Hand Patch | 353 |
| | 119 | Pavement Surface Maint. | 2,762 |
| | 108 | Flush Coat | 938 |
| 1983 | 102 | Level with Premix | 2,629 |
| | 108 | Flush Coat | 8,659 |
| | 111 | Temp. Hand Patch | 668 |
| | 119 | Pavement Surface Maint. | 1,322 |
| 1984 | 102 | Level with Premix | 3,110 |
| | 103 | Fill Cracks | 1,533 |
| | 111 | Temp. Hand Patch | 631 |
| | 119 | Pavement Surface Maint. | 1,000 |
| 1985 | 102 | Level with Premix | 29,714 |
| | 103 | Fill Cracks | 3,410 |
| | 104 | Spot Seal Patch | 2,828 |
| | 105 | Surface/Base Replace | 2,120 |
| | 111 | Temp. Hand Patch | 473 |
| 1986 | 102 | Level with Premix | 2,027 |
| | 103 | Fill Cracks | 9,973 |
| | 108 | Flush Coat | 9,717 |
| | 111 | Temp. Hand Patch | 1,284 |
| | 119 | Pavement Surface Maint. | 2,769 |

## APPENDIX C

TABLE C-1 - NDT TEST LOCATIONS

| | 1987 | | 1988 | | 1989 | |
|---|---|---|---|---|---|---|
| Milepost | EB | WB | EB | WB | EB | WB |
| 152.115 | | X | | X | | X |
| 152.126 | | X | | X | | X |
| 152.161 | | X | | X | | X |
| 152.179 | | X | | X | | X |
| 152.204 | | X | | X | | X |
| 152.229 | | X | | X | | - |
| 152.249 | | X | | X | | X |
| 152.290 | | X | | X | | X |
| 155.035 | X | | X | | | |
| 155.075 | X | | X | | | |
| 155.115 | X | | X | | | |
| 155.126 | X | | X | | | |
| 155.161 | X | | X | | | |
| 155.179 | X | | X | | | |
| 155.204 | X | | X | | | |
| 155.249 | X | | X | | | |
| 155.290 | X | | X | | | |
| 155.312 | X | | X | | | |
| 155.369 | X | | X | | | |
| 155.398 | X | | X | | | |

(TABLE C-1 - Continued)

| | 1987 | | 1988 | | 1989 | |
|---|---|---|---|---|---|---|
| Milepost | EB | WB | EB | WB | EB | WB |
| 155.412 | X | | X | | | |
| 155.445 | X | | X | | | |
| 155.463 | X | | X | | | |
| 155.490 | X | | X | | | |
| 155.516 | X | | X | | | |
| 155.544 | X | | X | | | |
| 155.575 | X | | X | | | |
| 155.757 | X | | X | | | |
| 155.7855 | X | | X | | | |
| 156.035 | | X | | X | | X |
| 156.075 | | X | | X | | X |
| 156.115 | | X | | X | | X |
| 156.126 | | X | | X | | X |
| 156.161 | | X | | X | | X |
| 156.179 | | X | | X | | X |
| 156.204 | | X | | X | | X |
| 156.229 | | X | | X | | X |
| 156.249 | | X | | X | | X |
| 156.290 | | X | | X | | X |

Note: Four more testings were done in 1988 in the WB direction at MP 156.45 and 156.50 respectively. Two were done on the travel lane and two on the shoulder.

## APPENDIX D

### TABLE D-1 - BACKCALCULATED LAYER MODULI FROM 1987, 1988, AND 1989 DEFLECTION DATA

| | Backcalculated Layer Moduli (ksi) | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | E1(AC) | | | E2(PCCP) | | | E3(CTB) | | | E4(Subgrade) | | |
| Location | 1987 | 1988 | 1989 | 1987 | 1988 | 1989 | 1987 | 1988 | 1989 | 1987 | 1988 | 1989 |
| 152.115 WB | 147 | 238 | 656 | 550 | 1,814 | 140 | 51 | 35 | 53 | 25 | 26 | 30 |
| 152.126 WB | 192 | 258 | 909 | 5,823 | 5,946 | 1864 | 51 | 50 | 53 | 17 | 22 | 30 |
| 152.161 WB | 227 | 227 | 11000 | 3,664 | 2,242 | 9000 | 44 | 35 | 75 | 27 | 29 | 18 |
| 152.179 WB | 248 | 242 | 167 | 11,700 | 9,500 | 11700 | 75 | 50 | 53 | 23 | 28 | 30 |
| 152.204 WB | 105 | 200 | 317 | 9,000 | 6,073 | 11700 | 75 | 50 | 53 | 21 | 23 | 26 |
| 152.229 WB | 329 | 140 | - | 5,958 | 2,365 | - | 75 | 35 | - | 22 | 19 | - |
| 152.249 WB | 207 | 233 | 1000 | 2,006 | 5,500 | 9000 | 53 | 50 | 25 | 20 | 24 | 20 |
| 152.290 WB | 117 | 170 | 677 | 9,000 | 1,700 | 3550 | 75 | 43 | 53 | 18 | 28 | 25 |
| 156.035 WB | 251 | 176 | 1300 | 2,000 | 2,000 | 2853 | 598 | 53 | 53 | 30 | 29 | 29 |
| 156.075 WB | 167 | 234 | 1000 | 9,000 | 9,000 | 309 | 75 | 75 | 75 | 26 | 22 | 15 |
| 156.115 WB | 179 | 358 | 1300 | 9,000 | 9,000 | 1330 | 75 | 75 | 53 | 28 | 15 | 15 |
| 156.126 WB | 194 | 1,000 | 1000 | 2,396 | 9,000 | 2752 | 53 | 75 | 60 | 29 | 15 | 28 |
| 156.161 WB | 232 | 235 | 1000 | 9,000 | 2,000 | 9000 | 75 | 75 | 100 | 24 | 16 | 22 |
| 156.179 WB | 292 | 100 | 487 | 9,000 | 9,000 | 1727 | 75 | 75 | 250 | 29 | 10 | 34 |
| 156.204 WB | 180 | 245 | 1000 | 9,000 | 9,000 | 9000 | 75 | 75 | 75 | 24 | 19 | 31 |
| 156.229 WB | 150 | 480 | 506 | 2,000 | 2,236 | 1563 | 75 | 53 | 81 | 28 | 23 | 19 |
| 156.249 WB | 251 | 1,000 | 1300 | 2,035 | 2,000 | 3358 | 75 | 150 | 35 | 29 | 20 | 27 |
| 156.290 WB | 188 | 188 | 1300 | 3,684 | 9,000 | 1545 | 53 | 75 | 35 | 31 | 13 | 25 |

## TABLE D-2 - PCC STRUCTURAL LAYER COEFFICIENTS FOR 1987, 1988, AND 1989

| Location | Crack Spacing | Modulus of Broken PCCP (Ksi) | | | PCCP Structural Layer Coefficients (ASSHTO) | | | PCCP Structural Layer Coefficients (NAPA) | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | 1987 | 1989 | 1988 | 1987 | 1988 | 1989 | 1987 | 1988 | 1989 |
| 152.115 WB | 6' x 4' | 550 | 140 | 1,814 | .36 | .44 | .24 | .124 | .18 | .079 |
| 152.126 WB | " | 5,823 | 1,804 | 5,946 | .44 | " | .44 | .27 | .27 | .186 |
| 152.161 WB | " | 3,664 | 9000 | 2,242 | " | " | " | .23 | .20 | .31 |
| 152.179 WB | " | 11,770 | 11,700 | 9,500 | " | " | " | .34 | .32 | .34 |
| 152.204 WB | 2' x 2' | 9,000 | 11,700 | 6,073 | " | " | " | .31 | .26 | .34 |
| 152.229 WB | " | 5,958 | - | 2,365 | " | " | " | .27 | .20 | - |
| 152.249 WB | " | 2,006 | 9,000 | 5,550 | " | " | " | .19 | .27 | .31 |
| 152 290 WB | " | 9,000 | 3,550 | 1,700 | " | " | " | .31 | .18 | .23 |
| 156.035 WB | 3' x 3' | 2,000 | 2,853 | 2,000 | " | " | " | .19 | .19 | .21 |
| 156.075 WB | " | 9,000 | 309 | 9,000 | " | " | " | .31 | .31 | .10 |
| 156.115 WB | " | 9,000 | 1,330 | 9,000 | " | " | " | .31 | .31 | .166 |
| 156.126 WB | " | 2,396 | 2,752 | 9,000 | " | " | " | .20 | .31 | .21 |
| 156.161 WB | " | 9,000 | 9,000 | 2,000 | " | " | " | .31 | .19 | .31 |
| 156.179 WB | " | 9,000 | 1,727 | 9,000 | " | " | " | .31 | .31 | .18 |
| 156.204 WB | " | 9,000 | 9,000 | 9,000 | " | " | " | .31 | .31 | .31 |
| 156.229 WB | " | 2,000.0 | 1,563 | 2,236 | " | " | " | .19 | .20 | .175 |
| 156.249 WB | " | 2,034.7 | 3,358 | 2,000.0 | " | " | " | .19 | .19 | .23 |
| 156.290 WB | " | 3,684.4 | 1,545 | 9,000.0 | " | " | " | .23 | .31 | .174 |

# APPENDIX E

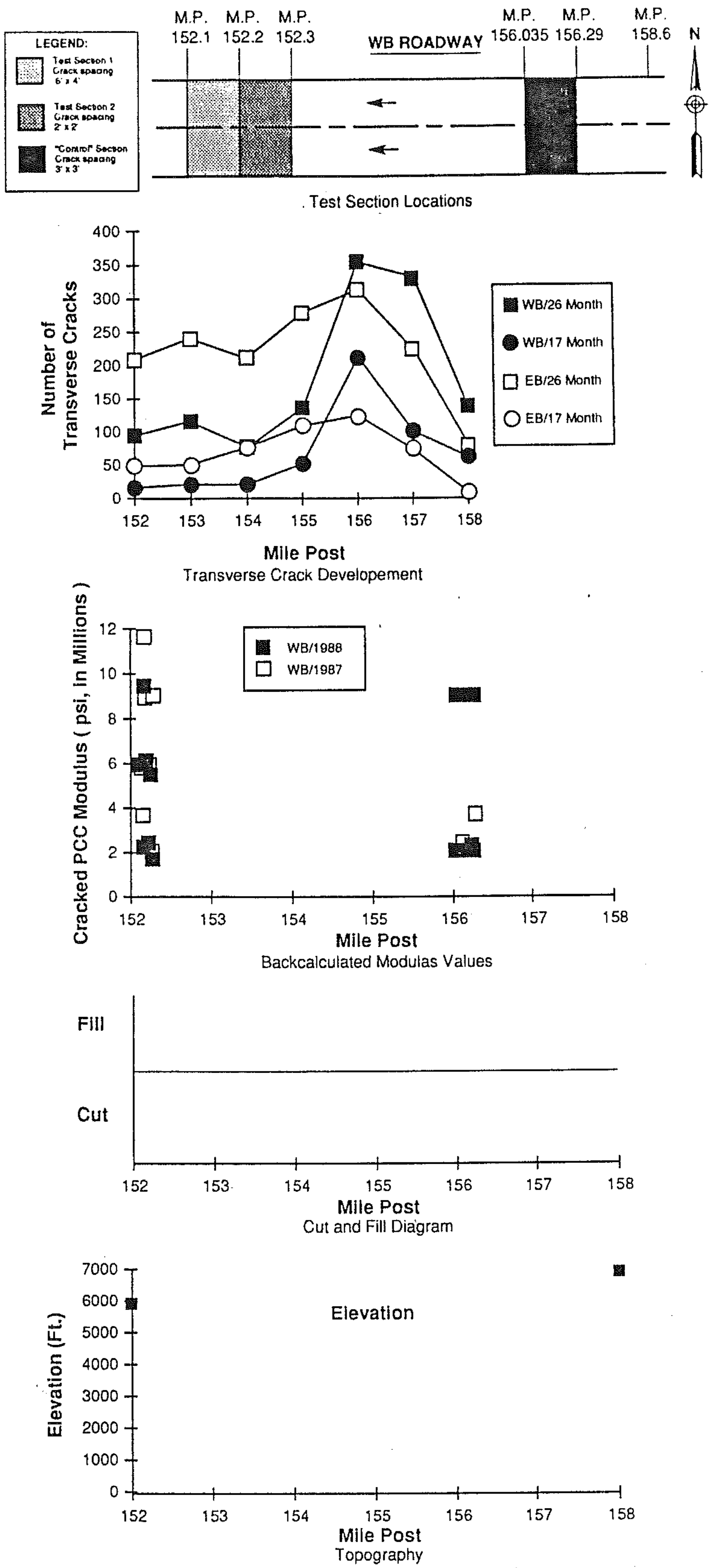


Figure E-1 - Characteristic Curves for the Project

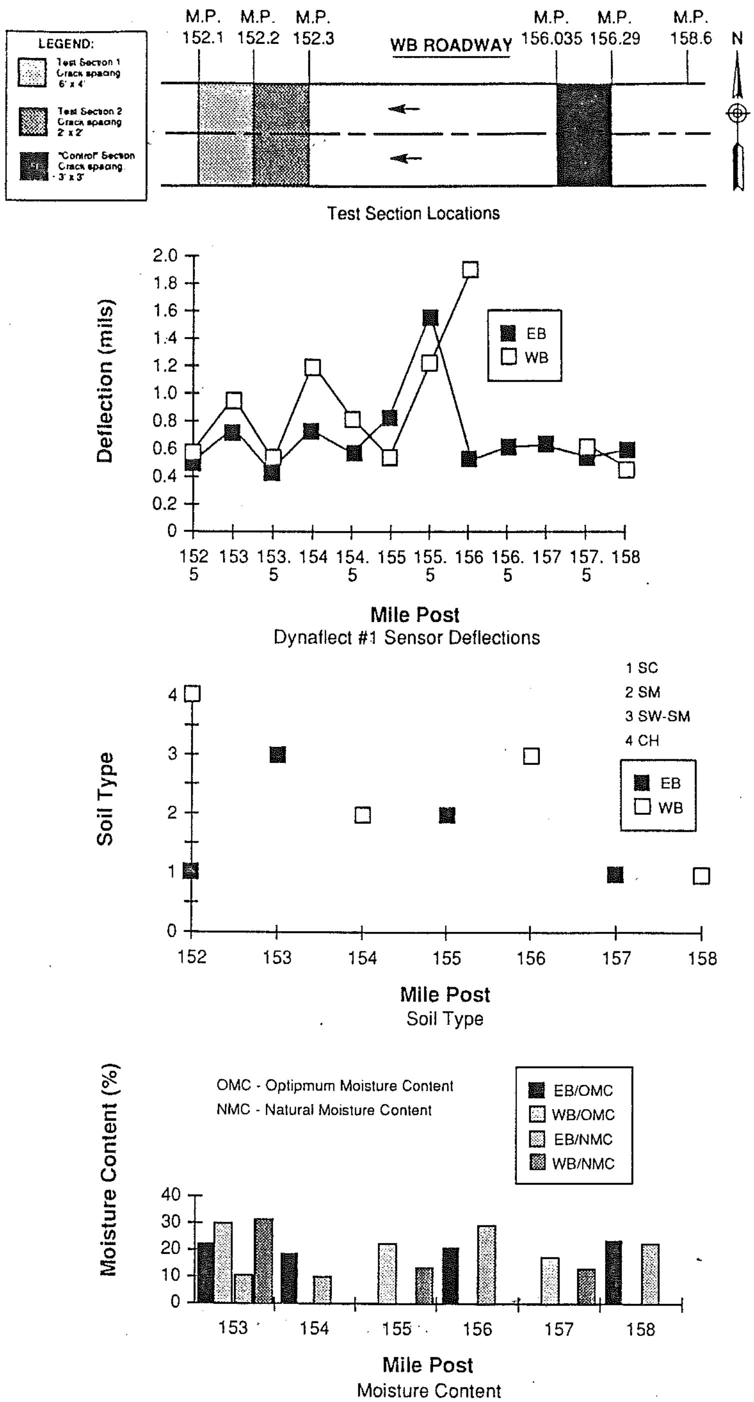


Figure E-1 (Con't) - Characteristic Curves for the Project