# AN EXPANDED EVALUATION OF ARIZONA'S TEN MILE CONCRETE TEST ROADWAY

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This paper describes the performance of the Superstition Freeway (State Route 360) located near Phoenix, Arizona,U.S.A.. The construction of this Freeway marked the beginning of a new era in the design philosophies of the Arizona Department of Transportation (ADOT) with regard to portland cement concrete pavements. It was the start of what became a ten-mile corridor of experimental test sections. The corridor consisted of seven sections constructed in one and two mile projects as part of the development of the Superstition Freeway. The experimental sections consist of:

- Full-Depth Jointed Plain Concrete Pavement (JPCP)
- JPCP Over Cement Treated Base (CTB)
- JPCP Over Lean Concrete Base (LCB)
- Prestressed Concrete Pavement Over LCB

These pavements have sustained 35% - 50% of their design loadings to date. Performance evaluations based primarily on roughness and skid characteristics are presented along with fault, deflection, and maintenance data. Limited comparisons are made between these sections and the earlier I-17/I-10 JPCP designs.

A statistical analysis of ADOT's pavement management inventory data was performed to develop linear models for both roughness and skid properties. The roughness models represent the expected roughness over the twenty year design life. The models representing skid properties could only be developed for the years in which field data was collected.

## INTRODUCTION

The Phoenix metropolitan area presents a unique environmental setting for the use of concrete pavements. Concrete surface temperatures of 130 F and relative humidities of 5%-10% are not unusual. The hot, dry climate adds difficulties to the construction of these pavements, requires greater attention to curing conditions, and induces greater warping and curling stresses. Conversely, the arid environment produces less pumping and faulting so often found elsewhere.

Historically State highways in Arizona, including those in the Phoenix area, were primarily constructed using asphalt concrete pavements. Only since the early 1960s, and the advent of the urban Interstate highways, have concrete pavements been utilized. Even today concrete pavements constitute only about three percent of Arizona's system.

The early ADOT freeway design (1961-1972) used for the construction of I-17 and I-10 in the Phoenix area consisted of 9 inches of JPCP over 3-4 inches of aggregate base course (ABC). During this era both fixed-form and slip-form techniques were used to pave two and three lanes at a time. Contraction joints were formed by metal inserts at 60 foot intervals followed by sawed joints at 15 foot spacings. These early joint designs did not use joint sealants, nor were they skewed. They were typically sawn 3/16 of an inch wide and 2 inches deep. Texturing was accomplished using a burlap drag.

Although representing only 54 center-line miles, the early I-17 and I-10 freeways were the only freeways serving the Phoenix urban area until 1972 when the first Superstition Freeway project was constructed.

## EXPERIMENTAL CORRIDOR

The Superstition Freeway is located within the Phoenix urban area in south-central Arizona. It extends from the junction of Interstate 10 approximately 26 miles to the east where it will eventually connect with US 60. The construction of this alignment began in 1970 and is still continuing today. Although it is not part

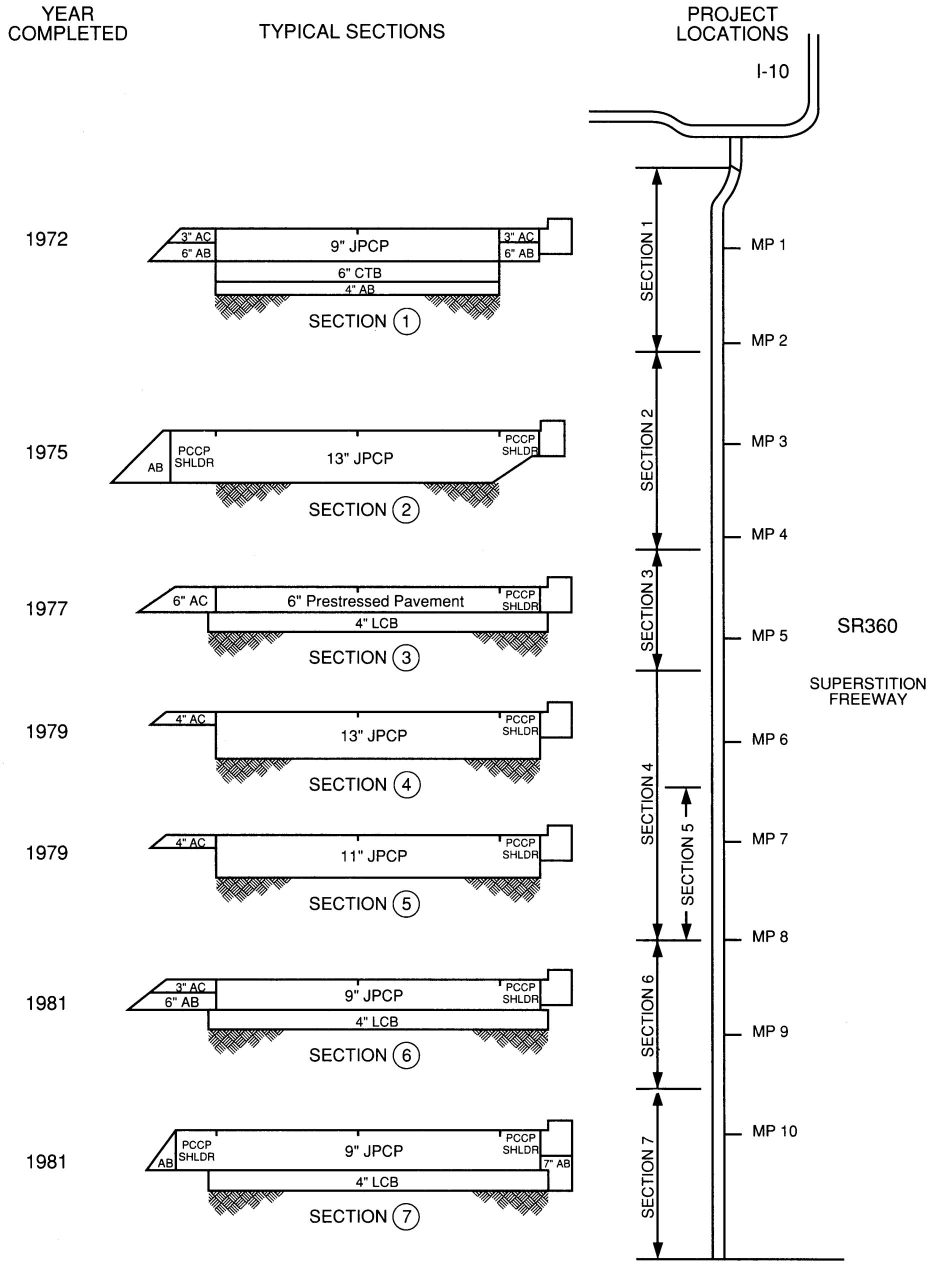


**Figure 1 – Superstition Experimental Sections**

of the Interstate System, it is designed and constructed to Interstate standards. Figure 1, Sections 1 through 7, indicates the alignment and typical section of each of the designs used in this experimental corridor.

EARLY I-17/ I-10 FREEWAY PERFORMANCE

By the early 1970s it had become apparent that significant problems existed with the performance of concrete pavements in the urban areas. Problems were noted with faulting, slab warping and curling, surface texture loss, and shoulder joint failures (1). Considerable problems with pavement growth had occurred and by 1971 pressure relief joints had been installed on approximately 30% of the system. Expansive forces were acting against abutment walls at overpasses to such an extent that some abutments had been deflected.

Maintenance on the asphalt concrete (A.C.) shoulders was a continual nuisance. Sealing of the shoulder joint and fog sealing the shoulder surface were frequently required. Also with the unusually high traffic growth, maintenance became more hazardous and expensive.

Table 1 indicates the various rehabilitation measures utilized on the earlier I-17/I-10 freeways in Phoenix. It is apparent that many of the concerns regarding the early design have been substantiated. From Table 1 it is evident that the early design has not failed due to fatigue and that distress has been manifested by loss of ride associated with joint related problems. Recent studies on an I-17 rehabilitation project found that approximately 40%-50% of the joints were distressed. Transverse joints formed with metal inserts had five times the incidence of distress that sawed joints exhibited (2,3,4). The typical rehabilitation for this early design consists of partial depth spall repair and grinding when Mays Meter readings reach 350-400 in./mi..

SUPERSTITION EXPERIMENTAL DESIGNS

It is important to note that throughout this paper, the term experimental section refers to "engineered experiments". The individual sections were constructed as one-of-a-kind experiments, without control sections, and often with more than one new feature incorporated.

Originally, this corridor was not intended as an experimental section. However, design improvements were recommended by ADOT and discussed in the literature at the time this segment was constructed (5,6,7). Since the Superstition was the only freeway under construction during that time, ADOT used the opportunity to implement new and innovative technologies. The result was the development of this experimental corridor. The various engineered experiments are shown in Table 2. It should be noted however, that not all the features shown in Table 2 are within the sections discussed in this paper. A third lane which was added to each of the experimental sections after their construction is included in this Table. The third lane is not evaluated in this paper.

TABLE 1 I-17/I-10 REHABILITATION SINCE CONSTRUCTION

| PAVEMENT SECTION | YEAR. CONST. | YEAR REHAB. | PER CENT OF SYSTEM | REHABILITATION TECHNIQUE |
|---|---|---|---|---|
| 9"JCPC/3-4"ABC | 1961-72 | 1971 | 30 | PressureRelieve Joints-Install Anchor Lugs |
| | | 1977 | 35 | Saw&Seal Joints |
| | | 1979 | 15 | Subseal Slabs |
| | | 1980 | 32 | Grind Pavements |
| | | 1985 | 15 | 3-Layer Overlay |
| | | 1986 | 12 | Grind Pavement |

TABLE 2 EXPERIMENTS ON THE SUPERSTITION FREEWAY

| EXPERIMENTAL FEATURE | DESCRIPTION |
|---|---|
| Pavement Thickness(similar types) | 11" & 13" JPCP |
| Pavement Types | JPCP(Full depth)<br>JPCP(CTB/ABC)<br>JPCP(LCB)<br>Prestressed/PostTensioned<br>CRCP(Lane Widening) |
| Texture Types | Burlap drag, Broomed, Transverse Tining |
| Drainage | Curb Edge Drain |
| Joint Types | Insert, Sawed |
| Joint Sealants | Silicone, Hot Poured |
| Shoulder Type | A.C., Concrete |
| Prestressed Slab Lengths | 400' & 500' |
| Weakened Plane Saw Depth | T/4 & T/3 |

Although there are many experimental features in this corridor, the projects constructed also had many similarities. They are listed as follows:

- -Constructed full width (two lanes)
- -Skewed joints sequentially spaced at 13,15,17,15 ft.
- -Same paving contractor and equipment (except section 1)
- -Similar Aggregate Sources
- -Constructed on level tangent alignment
- -Employed No. 4 bars mechanically placed by the paver @ 30 in. centers between travel lanes (except sec.3)
- -Constructed to a profile index, as measured by a California Profilograph of seven inches per mile or less
- -Maximum and minimum ambient summer temperature range is 103F-74F, while winter range is 67 F- 39 F.
- -Average precipitation is 7 inches per year with rainfall occurring on only 30-35 days. Fifty per cent of this rainfall occurs between November and March when the pavement joints are the widest.
- -In the design phase traffic was predicted to be 1% truck traffic, in actuality it was 2%-3%
- -Subgrades are predominantly AASHTO A6-A7
- -Concrete curbs were constructed adjacent to the outside shoulder to collect and divert runoff into catch basins
- -Sawed joints were sawn to a depth of T/4

The first three projects were designed with a pavement cross slope of 0.015 ft./ft. towards the outside shoulder. The remaining four sections utilized a cross slope of 0.02 ft./ft..

The seven sections within the experimental corridor each possessed one or more unique features, and typically represented a different design strategy. Table 3 indicates the pertinent data for each of these test sections.

## LONG-TERM PERFORMANCE EVALUATION

To date, approximately 10 years of performance history exists for the major portion of the Superstition Freeway and over 25 years for the early sections of the Phoenix urban freeways. In retrospect, the early pavement designs have long outlived their expected design traffic loadings. Table 4 indicates the estimated design loadings, estimated actual loadings, and the loadings which would be predicted by the 1986 AASHTO guide. It is evident that the early freeway design (1961-1972) has sustained approximately three times the loading predicted by current design procedures.

Table 5 indicates the type of rehabilitation, year rehabilitated, and per cent of roadway rehabilitated for each of the Superstition experimental sections.

TABLE 3 EXPERIMENTAL SECTION PROJECT DATA

| PAVEMENT Section | Project Length (MI.) | Texture Type | | | Trans. Jt.Type | | | Long. Jt.Type | | Max.Agg Size (IN.) | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Burlap Drag | Broomed | Tined | Metal Insert | Plastic Insert | Sawed | Plastic Insert | Sawed | 2.0" | 2.5" |
| Section 1 9"JPCP/6"CTB/4"AB | 1.8 | X | | | X | X | | X | | | X |
| Section 2 13"JPCP-1975 | 2.03 | | X | | | | X | | X | | X |
| Section 3 6"PRESTRESSED/LCB | 1.2 | | X | | | | | X | X | X | |
| Section 4 13"JPCP-1979 | 2.65 | | | X | | | X | | X | X | |
| Section 5 11"JPCP-1979 | 1.5 | | | X | | | X | | X | X | |
| Section 6 9"JPCP/4"LCB | 1.57 | | | X | | | X | | X | X | |
| Section 7 9"JPCP/4"LCB/DRN. | 1.02 | | | X | | | X | | X | X | |

TABLE 4 ESTIMATED TRAFFIC LOADINGS (ONE DIRECTION)

| PAVEMENT SECTION | DESIGN ESALs** | AASHTO ESALs** | ACTUAL ESALs** | LANE ADDED |
|---|---|---|---|---|
| 9"JPCP/3-4"ABC(1961-72)* | N.A. | 6 | 15-25 | |
| 9"JPCP/6"CTB/4"ABC | | 6.6 | 2.9 | 1984 |
| 13"JPCP/SUBGRADE | 7.1 | 7.4 | 2.8 | 1984 |
| 6"PRESTRESSED/4"LCB | 4.0 | *** | 2.5 | 1983 |
| 13"JPCP/SUBGRADE | 3.4 | 7.4 | 2.0 | 1983 |
| 11"JPCP/SUBGRADE | 3.6 | 2.6 | 2.0 | 1983 |
| 9"JPCP/4"LCB | 3.6 | 2.5 | 1.5 | 1988 |
| 9"JPCP/4"LCB&DRAIN. | 3.4 | 3.1 | 1.5 | 1988 |

* This is the early I-17/I-10 design, not a Superstition Project
** ESALs Expressed in Millions

TABLE 5 PAVEMENT REHABILITATION ON SUPERSTITION PROJECTS SINCE CONSTRUCTION

| PAVEMENT SECTION | YEAR CONST. | YEAR REHAB. | PER CENT OF SECTION | REHABILITATION TECHNIQUE |
|---|---|---|---|---|
| SECTION 1 9"JPCP/6"CTB/4"ABC | 1972 | 1986 | 100 | Groove Pavement Saw&Seal Joints |
| SECTION 2 13"JPCP/SUBGRADE | 1975 | 1986 | 100 | Groove Pavement |
| SECTION 3 6"PRESTRESSED | 1977 | 1978 | 100 | Joint Repair |
| SECTION 4 13"JPCP/SUBGRADE | 1979 | | | |
| SECTION 5 11"JPCP/SUBGRADE | 1979 | | | |
| SECTION 6 9"JPCP/4"LCB | 1981 | | | |
| SECTION 7 9"JPCP/4"LCB&Drain. | 1981 | | | |

## TRAFFIC HISTORIES

Figures 2 and 3 depict the characteristic traffic levels and growth rates for each of the typical pavement sections. It is evident that the I-10 and I-17 Freeways not only experience significantly higher traffic loadings than the Superstition Freeway, but that these loadings are increasing at a higher rate. This finding is particularly important when relating faulting experience for pavements deriving load transfer through aggregate interlock alone, as is the case for the Superstition projects, to designs for other traffic conditions. Previous research has suggested that for light commercial traffic, the expected pavement design thickness difference between a doweled and undoweled pavement is negligible (8). For high volumes of commercial traffic the thickness difference may be on the order of 19% of the total pavement section.

Figure 2 indicates that although slight differences exist between the Superstition sections, particularly in the initial growth rates of the later projects, they are reasonably similar and exhibit constant growth rates.

The authors have chosen to present most of the performance data in terms of time instead of ESALs. This is due to the questionable traffic data that exists historically. The addition of a third lane on each section after its construction further confounds the computation of the traffic data. Unfortunately, performance with time is difficult to apply to other conditions.

## PERFORMANCE CRITERIA

Pavement performance is characterized by the loss of serviceability or roughness,

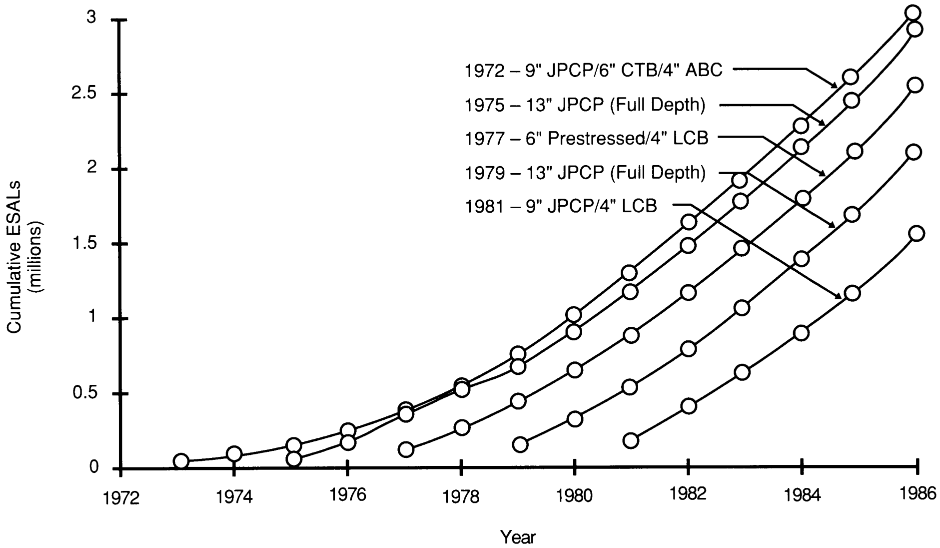


**Figure 2 – Characteristic Traffic Growth on Superstition Freeway**

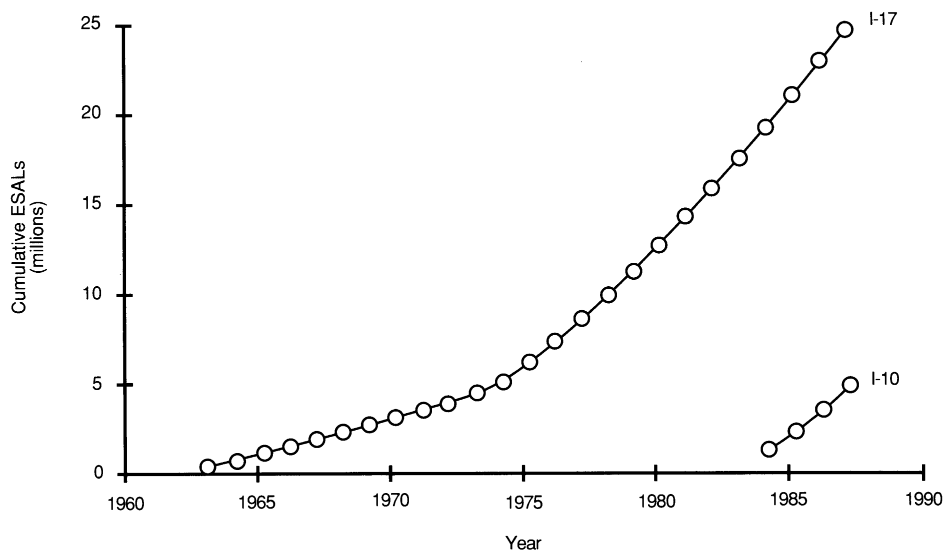


**Figure 3 – Characteristic Traffic Growth on I-17 and I-10**

and the loss of texture or skid properties. Although other properties such as faulting, joint efficiencies, and surface distress are discussed, the paucity of existing data precludes developing strong relationships with these performance variables.

INVENTORY DATA

Arizona performs an annual inventory of its highway network and collates 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 (PMS) 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 cardinal milepost location at which the readings began. Readings taken in the opposite roadway direction are then assigned to the corresponding cardinal mile posts. Mays ride data has been collected since 1972 in Arizona and since 1974 on the Superstition Freeway.

To ensure time stability of the data, ADOT employs extensive calibration procedures. During the inventory data collection, the Mays meter is tested weekly over 12-30 control sections to verify the calibration of the equipment. The data is normalized to the 1971 calibration values prior to inputting into the PMS. This provides consistent measurement standards with time. This is very important since ADOT has employed five Mays meters since 1971 and has replaced the car every two to three years.

Skid (friction) is determined by a Mu-Meter which is a continuous recording friction measuring trailer. The Mu-Meter measures the side-force friction generated by the test surface and two pneumatic tires set at a toe-out angle of 7 1/2 degrees. During the test, a 0.18 inch layer of water is placed under the test tires to simulate wet pavement conditions. Continuous readings are obtained for a five hundred foot interval. The readings are averaged and the results assigned to the cardinal milepost location which corresponds with the beginning of the test. Mu-Meter data have been collected since 1978.

During inventory data collection, the Mu-Meter is calibrated daily using a 4 ft. calibration board provided by the manufacturer. Additionally, the device is calibrated weekly over five test sections to verify its accuracy. Three Mu-Meters were utilized during the data collection on the Superstition.

Patching, cracking and faulting are determined for 1000 sq. ft. in the travel lane beginning at the milepost. The results are summarized and recorded at the corresponding cardinal milepost. These data have been available in the Arizona PMS database since 1981.

## DATA ANALYSES

Least square regressions were performed on the Mays Meter and Mu-Meter data from ADOT's PMS database to develop design life performance models. These models, based on 5 to 14 years of inventory data, were used to evaluate the predicted roughness of each of the sections throughout their twenty year design lives. Models representing the skid characteristics were developed for the years of available data. In this way, it is possible to compare the performance of various design strategies after only 35% to 50% of their design lives. Additionally, it is possible to predict expected future rehabilitation needs.

Although the shortcoming of extrapolating beyond the available data with regression equations is acknowledged, this approach was taken for two reasons. First, it is the only currently available method for comparing the performance of the various sections. Secondly, if the observed F-ratio is at least four to five times the F-table value, the model may be a useful predictor (9). This was the case for the data analyzed in this paper.

The results of these analyses are presented in Tables 6 and 7. The regression equations which provided the highest coefficient of determination without increasing the residual mean square were selected. The F-test was used to verify the linear hypothesis of each model. The t-test was used to identify the significance of the parameter coefficients. In all but one instance, a constant or a first order equation provided the best fit. In the exception, there were insufficient data to develop an adequate model.

The Mays Meter data indicated that four of the pavement sections were best modeled with a constant term, while three of the sections were best represented with first order equations. $R^2$ for the first order equations ranged between 0.61 and 0.99.

The Mu-Meter data indicated that the pavement sections textured with a burlap drag or nylon broom were best represented by a constant. Pavements textured by transverse tining were best represented by first order equations. $R^2$ for the first order equations ranged between 0.78 and 0.85

Paired t-tests were performed on the data to determine if any significant difference existed between the eastbound (EB) and westbound (WB) roadways. At a significance level of 0.05, two of the projects indicated a significant difference for roughness and three of the projects indicated a significant difference for skid. Therefore, to provide as large a database as possible, the eastbound and westbound data were combined to provide the best estimate of performance. The combined data were then used to develop the regression models.

## CHANGE IN PAVEMENT ROUGHNESS WITH TIME

Using the Mays Meter data from ADOT's PMS database, linear regression models were developed for each of the Superstition test sections as previously described. The models, shown in Figure 4, represent the predicted performance of each of the sections over their 20 year design lives. This includes both the predicted "as constructed" roughness as well as the predicted "future roughness" for each section.

The 20 year average roughness indicates the 13" full-depth sections perform the best with an average of 149 in./mi.. The composite pavements and the 11" full-depth section are next best with a predicted average of approximately 180-185 in./mi..

TABLE 6 MAYS METER DATA ANALYSES

| SECTION NO. | PAVEMENT | $R^2$ | EQUATION | N |
|---|---|---|---|---|
| 1 | 9"JPCP/6"CTB/4"ABC | 0.96 | Y=65+14X | 11 |
| 2 | 13"JPCP/SUBGRADE | 0.81 | Y=73+5X | 13 |
| 3 | 6"PRESTRESSED/4"LCB | 0.0 | Y=220 | 11 |
| 4 | 13"JPCP/SUBGRADE | 0.0 | Y=149 | 9 |
| 5 | 11"JPCP/SUBGRADE | 0.0 | Y=181 | 7 |
| 6 | 9"JPCP/4"LCB | 0.62 | Y=143+3.4X | 7 |
| 7 | 9"JPCP/4"LCB@DRAIN | 0.0 | Y=163 | 6 |
| * | 9"JPCP/4"LCB | 0.99 | Y=11X | 5 |

* Later Superstition Project not shown in Figure 1
Note: X= Number of years since 1970

TABLE 7 MU-METER DATA ANALYSES

| SECTION NO. | PAVEMENT | $R^2$ | EQUATION | N |
|---|---|---|---|---|
| 1 | 9"JPCP/6"CTB/4"ABC | 0.0 | Y=40.7 | 7 |
| 2 | 13"JPCP/SUBGRADE | 0.0 | Y=43.3 | 7 |
| 3 | 6"PRESTRESSED/4"LCB | 0.0 | Y=43.5 | 7 |
| 4 | 13"JPCP/SUBGRADE | 0.84 | Y=106-3.6x | 8* |
| 5 | 11"JPCP/SUBGRADE | 0.78 | Y=97-3.3X | 7 |
| 6 | 9"JPCP/4"LCB | 0.85 | Y=104-3.3X | 5 |
| 7 | 9"JPCP/4"LCB&DRAIN | 0.85 | Y=110-3.4X | 5 |

Note: X= Number of years since 1970
* Includes 1988 Data

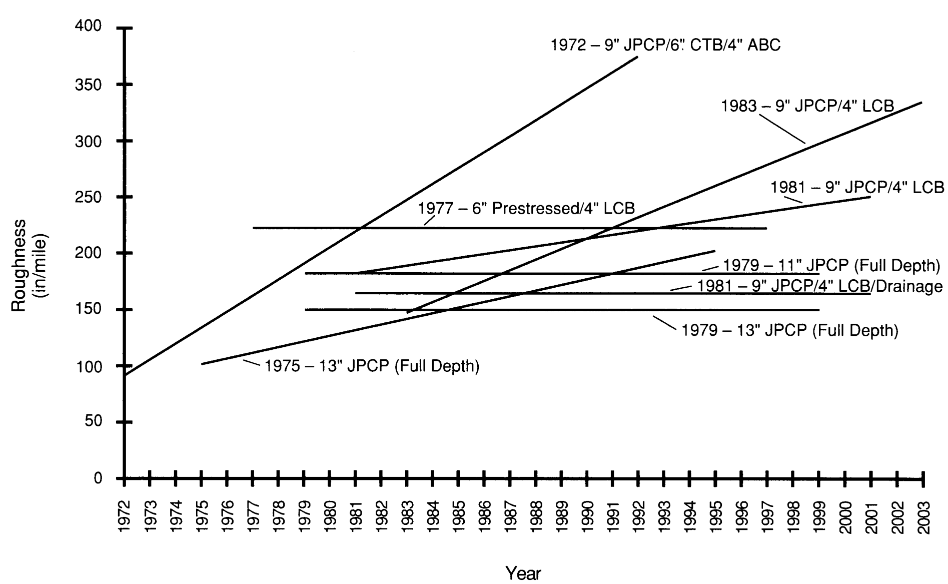


**Figure 4 – Twenty Year Regression Models for Roughness**

The prestressed and 1972 "control" section follow with 220 and 233 in./mi., respectively.

The performance of each of the sections listed in Figure 1 is summarized in Table 8. For each section, the predicted initial roughness at the time of construction, the predicted average 20 year roughness, the predicted current roughness, and the predicted rate of change of roughness as a function of time are shown.

The data in Table 8 and Figure 4 indicate that the first experimental section was constructed smoother than the other sections. The only pavement which approached this level of construction smoothness was the 1975 13" full-depth pavement. The 1979 13" full-depth section was constructed considerably rougher. Since Section 1 (9"JPCP/6"CTB/4"ABC) did not utilize concrete shoulders, it may have been easier for the contractor to pave the narrower width and hence deliver a smoother project.

The composite pavements (sections 6,&7) exhibited approximately 3 in./yr./mi. and 0 in./yr./mi. increase, respectively. Again this is a significant difference in performance. However, a pavement of similar design, constructed in 1984 exhibited a roughness increase of 11 in./yr./mi.. This is exceptional for this pavement type and may be attributable to random cracking which occurred on this project during hot weather construction. It does, however, indicate the potential for large variations in performance of similar designs.

The 1979 13" full-depth section has two miles of roughness data within the section. The data obtained between mileposts six and seven describes the current roughness as approximately 150 in./mi.. The continuous data collected between mileposts seven and eight describes the current roughness as approximately 190 in./mi.. Surprisingly, the 150 in./mi. section is equivalent to the smoothest section discussed (1975-13" full depth), while the 190 in./mi. section is comparable to the rougher sections.

TABLE 8 PREDICTED MAYS ROUGHNESS DATA

| PAVEMENT SECTION | ASBUILT ROUGHNESS (IN./MI) | AVE. 20YR ROUGHNESS (IN./MI.) | CURRENT ROUGHNESS (IN./MI.) | CHANGE IN ROUGHNESS IN/MI/YR |
|---|---|---|---|---|
| 9"JPCP/6"CTB/4"ABC | 93 | 233 | 263 | 14 |
| 13"JPCP/SUBGRADE | 98 | 148 | 157 | 5 |
| 6"PRESTRESSED/4"LCB | 220* | 220* | 220* | 0* |
| 13"JPCP/SUBGRADE | 149 | 149 | 149 | 0 |
| 11"JPCP/SUBGRADE | 181 | 181 | 181 | 0 |
| 9"JPCP/4"LCB | 180 | 214 | 201 | 3 |
| 9"JPCP/4"LCB@DRAIN. | 163* | 163 | 163* | 0* |
| 9"JPCP/4"LCB** | 140 | 252 | 190 | 11 |

* Data is tainted due to inventory collection methods
**Later Superstition Project not in Experimental Section

The prestressed pavement (section 3) was constructed significantly rougher than all other sections. This is probably due to the block outs necessary every four hundred feet to accommodate the gap slabs. Although production rates of 5600 sq. yds. per day were obtained, it is difficult to believe that the problems attendant to paving through these gab slab areas did not significantly contribute to the roughness. The joint design used has incurred considerable maintenance problems since construction.

The 1975 and 1979 13" full-depth sections exhibited roughness increases of 5 in./yr./mi. and 0 in./yr./mi., respectively. This is a large variation for pavements of similar design. The only differences were the tapered shoulder and anchor slab utilized in the 1975 design and the transverse texture utilized in the later design.

This wide disparity suggests the importance of large sample sizes and the significant variability attributable to construction quality.

## JOINT FAULT DATA

In 1987, six of the seven sections were independently surveyed (10). The survey evaluated a 1000 foot section within each of the experimental projects. The results of this survey are shown in Table 9.

The fault data indicates the best performing sections are the 13" full-depth sections followed closely by the composite section without drainage. The poorest performing section is the first experimental section. The 11" full depth and the composite section with edge drainage have performed similarly. The significant difference in fault development

TABLE 9 FAULT DATA (10)

| PAVEMENT SECTION | FAULT RATE (IN./ESALs*) | AVERAGE TRANSVERSE FAULTING(IN.) |
|---|---|---|
| 9"JPCP/6"CTB/4"ABC | 0.027 | .08 |
| 13"JPCP/SUBGRADE(1975) | 0.004 | .01 |
| 13"JPCP/SUBGRADE(1979) | 0.005 | .01 |
| 11"JPCP/SUBGRADE | 0.015 | .03 |
| 9"JPCP/4"LCB | 0.007 | .01 |
| 9"JPCP/4"LCB&DRAIN | 0.018 | .02 |

* ESALs Expressed in Millions

TABLE 10 RESULTS OF FWD TESTING

| PAVEMENT SECTION | MID-SLAB DEFLECTIONS (MILS) | CORNER DEFLECTIONS (MILS) | LOAD TRANSFER EFFICIENCIES (%) |
|---|---|---|---|
| 9"JPCP/6"CTB/4"ABC | 3.3 | 8.6 | 94 |
| 13"JPCP/SUBGRADE | 2.7 | 4.6 | 100 |
| 13"JPCP/SUBGRADE | 2.3 | 6.3 | 100 |
| 11"JPCP/SUBGRADE | 3.2 | 12.5 | 100 |
| 9"JPCP/4"LCB | 2.1 | 5.0 | 100 |
| 9"JPCP/4"LCB@DR. | 2.8 | 6.9 | 94 |

between the composite section with and without drainage suggests careful evaluation in the future. It is possible that collecting moisture at the shoulder edge is contributing to fault development.

There is reasonable correlation between the ranking of the pavements by both fault data and Mays Meter data. The exception is the composite pavements which have reversed ranking for faulting and roughness. All the pavements have minimal faulting at this time.

RESULTS OF NONDESTRUCTIVE TESTING

In November of 1987, deflection tests were conducted on all but the prestressed section (10). The tests were conducted with a Dynatest 8002 FWD and a loading of 12000 lbs. utilized for the results reported herein.

For all but the first experimental section, which utilized A.C. shoulders, a minimum of seven slab centers and seven slab edges (@mid-span) were tested. Also, fourteen slab corners were tested at the joint, including load transfer across the transverse joint.

For the section with A.C. shoulders (section 1) ten slab centers and twenty slab corners, including load transfer across the transverse joint, were tested. The results of the FWD testing are shown in Table 10.

The results of the FWD testing indicate that the composite section without drainage and the 1975 13" full-depth section have the best deflection characteristics. Typical mid-slab deflections were 2.1-2.7 mils, while typical corner deflections were 4.6-5.0 mils for these pavements.

When evaluating load transfer and joint deflections, it should be noted that the 1975 13" full-depth section is the only pavement which utilized terminal anchor slabs in the Superstition corridor. This section also utilized a tapered shoulder (13" to 6").

The 11" full-depth section had the worst deflection characteristics. Although load transfer at the joint is still good, the deflections are large enough to suspect the presence of voids beneath the joints.

The significant difference between the composite sections with and without drainage is paradoxical. The curb drains appear to have increased both faulting and deflections. It seems quite likely that the presence of the drains is decreasing the subgrade support by allowing an increase in moisture.

The sections which demonstrated the largest corner deflections also exhibited the highest rate of faulting. If only deflection characteristics and faulting are considered, the poorest performing sections are the first experimental section, representative of earlier design strategies, the 11" full-depth section, and the composite section with drainage. The best performing sections are the two 13" full-depth sections and the composite section without drainage.

The load transfer efficiencies determined with the FWD testing equipment are significantly higher than those reported from testing conducted with Dynaflect equipment in 1980 by ADOT (1). Efficiencies this high are surprising for pavements which rely on aggregate interlock alone.

CHANGE IN PAVEMENT SURFACE TEXTURE WITH TIME

Surface texture, as discussed in this paper, refers to pavement surface characteristics measured by the Mu-Meter. Three surface texturing techniques were utilized; longitudinal burlap drag, longitudinal brooming, and transverse tining.

Figures 5 and 6 indicate the change in skid properties for each of the sections as a function of traffic loading and roadway direction. As evident in these figures, the transverse tining results in significantly higher values at the time of construction, typically around 70. The

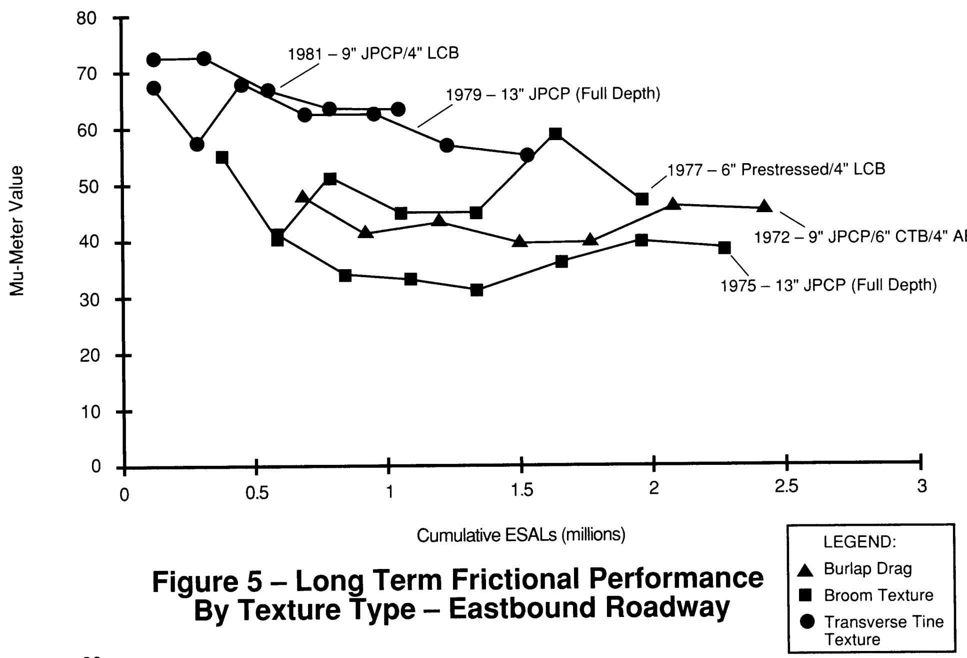


**Figure 5 – Long Term Frictional Performance By Texture Type – Eastbound Roadway**

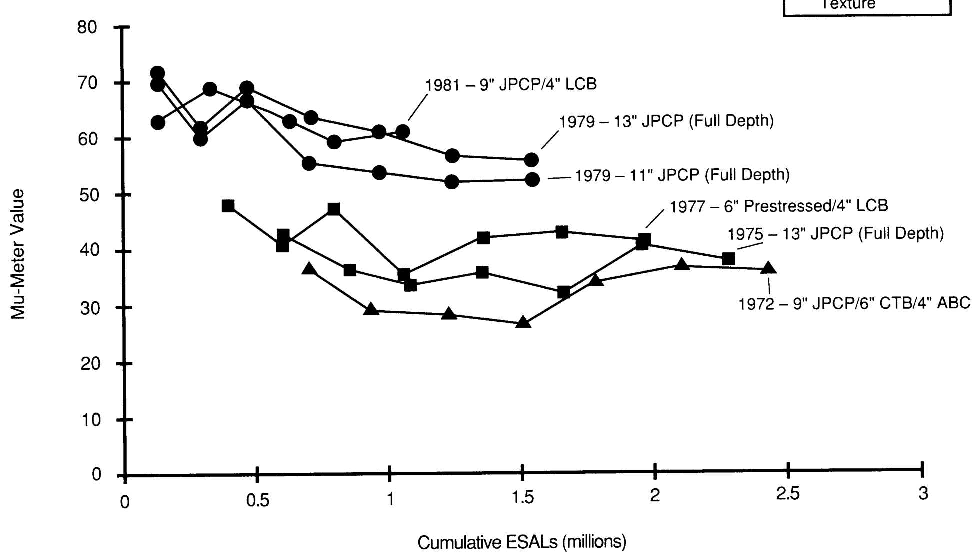


**Figure 6 – Long Term Frictional Performance By Texture Type – Westbound Roadway**

skid properties of the tined texture rapidly deteriorate until approximately 1.5 million ESALs when they appear to stabilize at a value of 50-55. The broom textures typically attained a construction skid property near 40, then remained relatively constant at this value. The burlap drag texture performed similarly to the broom texture. Since only one project had a burlap drag texture, it is difficult to compare the results since different results are evident for the EB and WB roadways.

Figure 7 indicates the regression models which were developed from ADOT's Mu-meter data for each of the sections for the years transverse texture performance is markedly different from what would be expected. This results from data collection problems between 1985 and 1987 due to faulty equipment. The authors believe that data from this period would have developed nonlinear models similar in shape to the conceptual model indicated in Figure 7. The limited data prevented development of reasonable 20 year regression models.

Table 11 summarizes the predicted initial skid and predicted rate of change per year for each of the texture types.

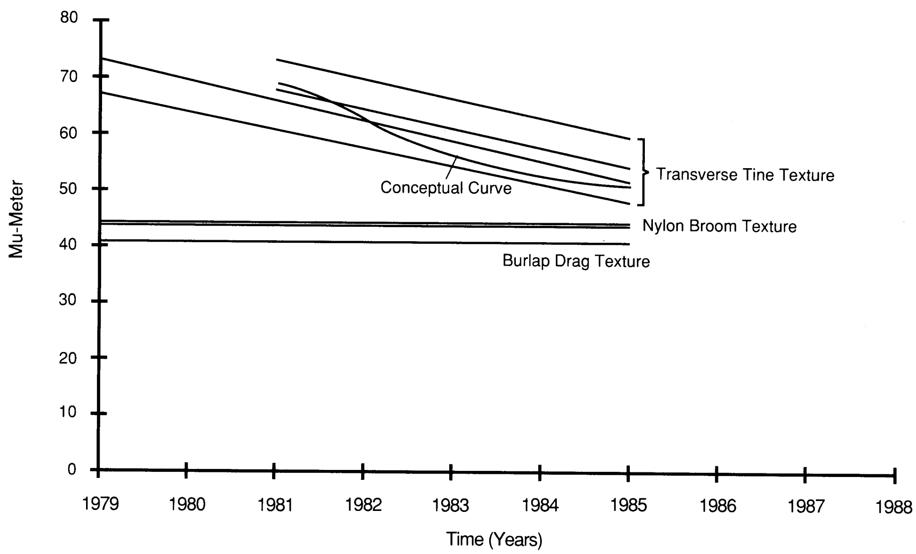


**Figure 7 – Mu-Meter Regression Lines for Years of Available Data**

TABLE 11 CHANGE IN SKID WITH TIME

| PAVEMENT SECTION | INITIAL SKID | RATE OF CHANGE (UNITS/YR) | TEXTURE TYPE |
|---|---|---|---|
| 9"JPCP/6"CTB/4"ABC | 40.7 | 0 | Burlap Drag |
| 13"JPCP/SUBGRADE | 43.3 | 0 | Long.Broom |
| 6"PRESTRES/4"LCB | 43.5 | 0 | Long. Broom |
| 13"JPCP/SUBGRADE | 72 | -03.6 | Trans.Tining |
| 11"JPCP/SUBGRADE | 68 | -03.3 | Trans.Tining |
| 9"JPCP/4"LCB | 68 | -03.3 | Trans.Tining |
| 9"JPCP/4"LCB&DRAIN | 74 | -03.4 | Trans.Tining |

of available data. It should be noted that the regression models are a function of time and not loading as was the case in Figures 5 and 6 above. These models suggest that the burlap and broom textures produced constant skid properties of 41 and 43, respectively. The transverse textures have beginning values of 75-85 and then decrease approximately three units per year. This constant linear decline in

## MAINTENANCE COST FOR SECTIONS

ADOT uses a maintenance management system to track approximate costs expended for the highway network. Each of the specific activities performed by maintenance personnel are recorded by route and milepost. Work performed on a given route is recorded by the beginning and ending mileposts for each day. The management system distributes the reported costs evenly over one tenth mile increments between the recorded mileposts for the activities performed.

Figure 8 indicates the maintenance cost for each of the sections for each year since construction. The dollars have been standardized to 1982 dollars using the fixed weighted price index (11). In viewing this data, it should be noted that the process of averaging all activities performed between the beginning and ending mileposts of the work day decreases the reliability of this information for analyzing short sections such as a mile or less. This information has only been included to demonstrate trends.

CONCLUSIONS

The seven experimental sections were successfully constructed and are performing satisfactorily after approximately ten years of service.

The first Superstition project, similar to the early I-17/I-10 designs, has the poorest performance of all the sections. Even so, it is performing with little faulting or distress. As with the early I-17/I-10 design, this strategy may require grinding near its twenty year design life.

The two 13" JPCP (Full Depth) sections are the best performing design. They have an expected average roughness of 150

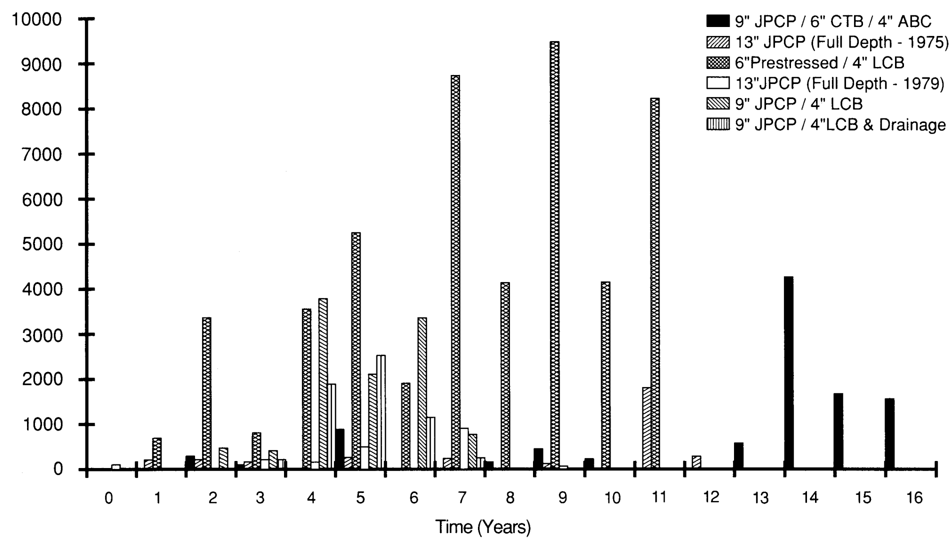


**Figure 8 – Annual Maintenance Costs/Mile Versus Time Since Construction**

Figure 8 clearly demonstrates the higher maintenance costs attendant to the prestressed pavement section. It also indicates that the composite pavement sections have a significantly higher average cost than the other sections. This appears contrary to the actual maintenance on the composite pavement sections. However, erosion of the adjacent slopes did occur on these sections about the time the high maintenance costs were incurred. It is believed that the repair of the erosion damage is included in this total cost and is providing misleading information on the composite sections.

The projects in the Superstition corridor began incurring maintenance expenditures approximately three to six years after construction. This is similar to the projects on I-17 and I-10.

in./mi. with almost no faulting. No rehabilitation is anticipated during their design life. ADOT is currently using this design concept with a granular base on routes which do not have heavy truck traffic.

The composite pavement sections demonstrated the greatest variability in performance with expected roughness ranging from 163 to 252 in./mi.. This design strategy may be more susceptible to construction quality and/or environmental effects.

Although the post-tensioned, prestressed pavement has maintained a constant roughness with time, it was constructed rougher than expected. This section has incurred maintenance in excess of what ADOT considers satisfactory. The project did demonstrate the viability of this technology and its potential applications. If a better joint design had been used and a smoother project constructed, the prestressed pavement would probably rank high.

Transverse tining demonstrated better skid properties than the longitudinal burlap drag or nylon brooming. Tine texturing appears to deteriorate significantly in the first several years and then approaches a constant value approximately ten units higher than that expected with a burlap drag.

The variability of construction quality suggests that pavement roughness is not readily assessed by short test sections unless a statistically adequate sample of sections is obtained.

The continued monitoring of these sections will validate or improve the regression models presented. As these projects approach their design lives they will provide valuable insight into the most cost effective designs in this hot, dry Arizona environment.

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