Report No. CDOT-DTD-R-93-8

Rehabilitation of Rutted Asphalt Pavements Project IR-25-3(96)

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Final Report
June 1993

Prepared in cooperation with the
U.S. Department of Transportation
Federal Highway of Administration

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Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
CDOT-DTD-R-93-8		

4. Title and Subtitle	5. Report Date
Rehabilitation of Rutted Asphalt Pavements Project IR-25-3(96)	May 1993
	6. Performing Organization Code File 11.075 1571P
7. Author(s) Ahmad Ardani	8. Performing Organization Report No.
9. Performing Organization Name and Address Colorado Department of Transportation 4201 E. Arkansas Avenue Denver, CO 80222	10. Work Unit No. (TRAIS) 11. Contract or Grant No.
12. Sponsoring Agency Name and Address Colorado Department of Transportation 4201 E. Arkansas Avenue Denver, CO 80222	13. Type of Report and Period Covered Final Report 14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in Cooperation with the U. S. Department of Transportation
Federal Highway Administration

16. Abstract

This report describes the testing, construction, and 5 years of performance evaluations of a rehabilitated rutted asphalt pavement. The primary objective of this study was to investigate the effectiveness of a 2-inch high stability overlay topped with a plant-mixed seal coat (PMSC) in alleviating the rutting distress on I-25 north of Denver in Colorado. The following is a brief description of some of the findings:

High-stability asphalt is not resistant to rutting, and rutting is returning to the rehabilitated southbound lanes, but at a slower rate than the standard mix.

A preconstruction examination of the existing pavement indicated that the top mat makes the greatest contribution to the total rutting.

The results of laboratory tests were largely inconclusive. No correlations were found between the measured rutting and pavement parameters such as stability, asphalt content, gradations, voids, etc.

Implementation: The results of this study demonstrated that high-stability pavement topped with a plan-mixed seal coat is not significantly better than the standard mix. In general, the high-stability mix could not eliminate the formation of rutting; however, it was able to slow down the rate of rutting.

17. Key Words	18. Distribution Statement
Rutting, High-stability, Overlays, Gradation Performance	No Restrictions: This report is availabale to the public through the National Information Service, Springfield, Virginia 22151

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	54	

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Project IR 25-3(96)

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I. Literature Review and Problem Discussion

Flexible pavement rutting has been a major problem in Colorado and many other states, especially on routes subjected to heavy truck or high-traffic volumes. In general, Colorado's flexible pavements are designed for a 20-year life expectancy; however, some of these pavements lose their rideability and become prematurely distressed before reaching their design life. Furthermore, the primary mode of failure has been in the form of rutting. Significant rutting (ruts in excess of 1 inch) can pose safety hazards to the travelling public and may eventually lead to major structural failure.

Rutting problems generally fall into the following four categories:

1- Consolidation or the vertical deformation of the upper-layer pavement due to traffic loadings.

2- Inadequate compaction.

3- Unsuitable aggregate.

4- Insufficient mix stability or instability caused by the stripping of asphalt below the surface (1).

Highway engineers are well aware of the increase in numbers of heavy trucks carrying heavier loads and some know that tire pressures have increased significantly. A few years ago, tire pressures of 75 PSI were considered reasonable for design, but recent surveys showed tire pressures in Texas averaging 110 PSI with a maximum of 155 PSI and in Illinois averaging 96 PSI with a 130 PSI maximum (2). Based on a study conducted in 1989, the average truck tire pressure in Colorado appeared to be 100 PSI, with a maximum of 150 PSI and with many exceeding 120 PSI (3).

Deep ruts can also result from lateral plastic flow of the mixture in the wheel paths. The primary reason for the occurrence of plastic flow is the excessive use of asphalt cement. Too much asphalt cement in a mixture reduces the friction between aggregates. The asphalt cement then flows under repeated load stresses causing permanent deformation. In contrast, mats with asphalt content lower than optimum will be rut and flush resistant, but will be prone to raveling and cracking. According to reference 1, most western states are willing to accept some cracking and deal with it through normal maintenance practices in order to reduce rutting which cannot effectively be handled by maintenance.

Plastic flow is also affected by the sizes, shapes, and texture of the aggregates in a bituminous mixture. Aggregate that is rounded is more easily displaced by an applied load. Aggregate that is angular (usually produced by crushing) will have a greater degree of interlock when a load is applied and will be more resistant to displacement by the load (4).

A number of recent studies suggest a definite correlation between rutting and material passing No. 200 sieve. A certain amount of dust (P200) is necessary in the mix to provide stiffness and improve workability. On the other hand, high percentages of P200 (dust to asphalt ratio exceeding the FHWA guideline of 1.2) overfill the voids and can cause flushing and eventually lead to rutting. Reference 5 recommends for CDOT to consider limiting the amount of P200 in mixtures used on higher-volume pavements. This could take the form of either a maximum dust to asphalt ratio or a maximum permissible P200.

Rutting is also controlled by the percent voids in a mix. In general, most of the recent studies have indicated a direct relationship between rutting and the amount of air voids in a mix. According to a recent publication by Colorado DOT, "Investigation of The Rutting Performance of Pavements in Colorado", air voids in the wheel path of 3.0% clearly distinguished pavements with good and bad rutting performances. Pavements with air voids less than 3% rutted and pavement with air voids greater than 3% did not rut from plastic flow (6).

II. Objectives

The Primary objective of this study was to evaluate and monitor the performance of a 2-inch high stability overlay used to rehabilitate the rutted pavement on I-25 southbound between MP 235 and MP 245. The evaluation consisted of the following:

1- A pre-construction examination of the existing rutting of the four-year old pavement.

2- Post-construction examination of the high-stability pavement for rutting, cracking, and stripping.

3- Estimate the useful life of the rehabilitation, and examine its practicability for future work.

III. Background

During the 1983 and 1984 construction seasons, ten miles of the I-25 southbound and northbound between MP 235 and MP 245 was overlaid with five inches of HBP Gr E. Despite a newly required Hveem stability of 37, the pavement started to show early rutting. The extent of the rutting was to the point that three years after construction the pavement rutting was averaging approximately 1/2 inch in the driving lanes with half this amount in the passing lanes. Some areas were rutted up to 1 inch. Cores taken from the site showed that the majority of the rutting was occurring in the top 1-1/2 inches of the overlay.

Two rehabilitation options were considered for this roadway:

- a thick asphalt overlay; and
- a thick, unbonded concrete overlay.

Although a concrete overlay was expected to be more durable, the high initial cost and the need for diverting traffic onto a detour during construction favored the selection of a thick asphalt overlay at this site (7).

The pavement design approach used for this project is based on the guidelines published by the Asphalt Institute in a report entitled "Asphalt Overlays for Heavy-trafficked PCC pavements" (IS-117). It consisted of a five-inch overlay over a nominal 1/2-inch leveling course. The specifications for the overlay mix are shown in Appendix A.

In 1985, a stretch of Interstate 25 immediately to the north of this project was rehabilitated with a 7-1/2 inch unbonded concrete overlay.

IV. Rehabilitation of Rutted Asphalt Pavements

A. Site Description

The subject project is located on a gently rolling terrain approximately 45 miles north of Denver between mileposts 235 and 240, and was built in the early 1960's (see Figure 1). The five-mile roadway was experiencing an Average Daily Traffic (ADT), in excess of 29,000 vehicles with 15 percent of that volume consisting of heavy vehicles. The present day ADT for this location is 41,400 vehicles with 16 percent heavy vehicles. This translates into an average daily truck traffic (ADTT) of over 6600 per day.

The four-lane highway had concrete slabs that were eight inches thick and constructed on an untreated aggregate base (7). Joints were plain and non-reinforced, with transverse joints spaced every 20 feet.

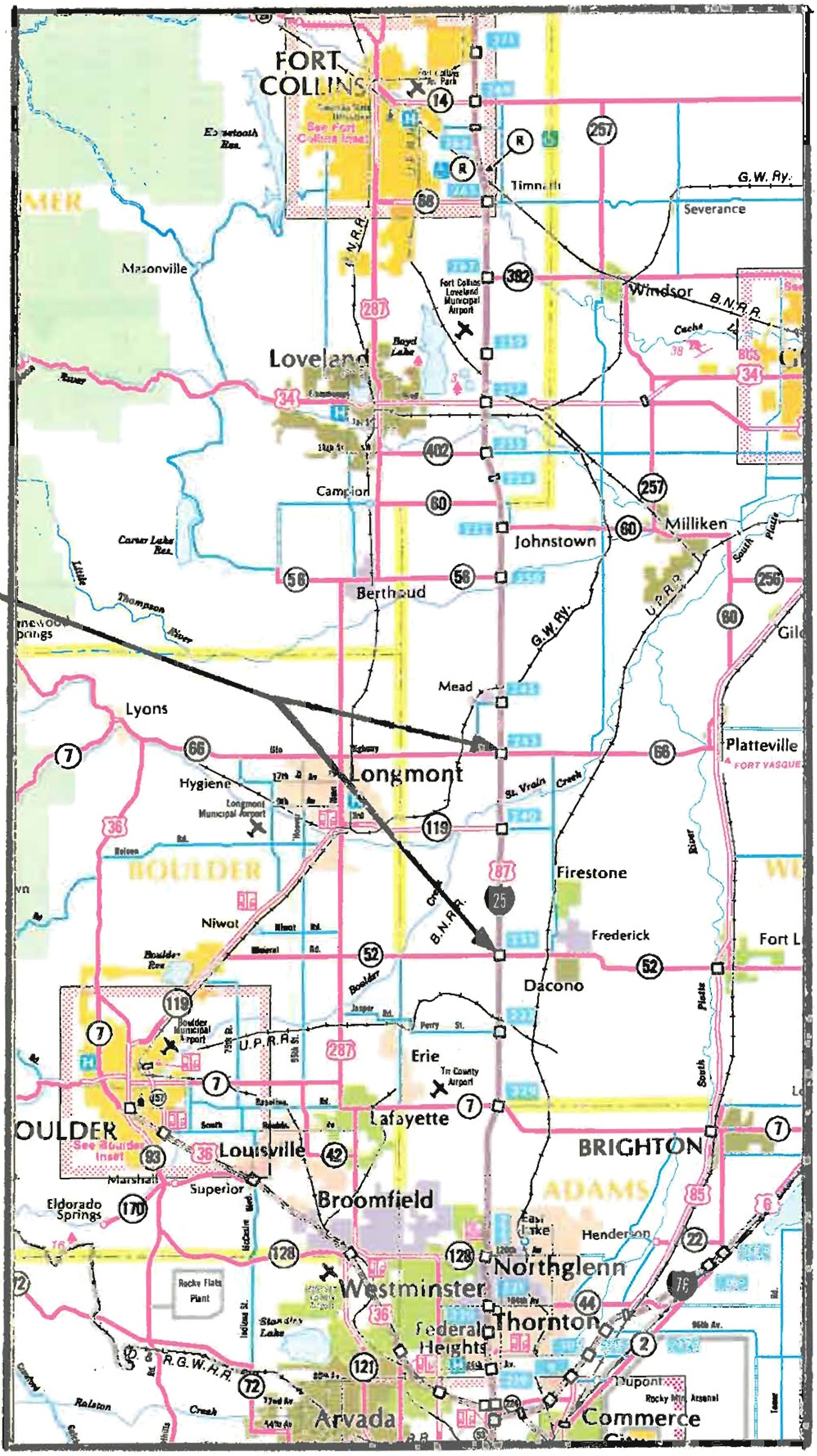
There was no load transfer across these joints except that provided by the aggregate interlock at the broken joint faces (7). The roadway had ten-foot outside and four-foot inside asphalt shoulders (three inches thick). A review of the existing pavement showed that approximately 90 percent of the slabs in the driving lane were cracked and the serviceability index of the pavement was rapidly declining.

The passing lane was in much better shape than the driving lane. Only 3 percent of the slabs in the passing lane were cracked. Only the critically unstable and broken slabs in the driving lane required removal and replacement.

B. Pre-construction examination

Figure 1

IR 25-3(78)
PROJECT LIMITS



The primary mode of failure in the existing pavement appeared to be rutting. In general, the rutting was more severe on the southbound direction of the interstate and in the driving lane. The wheel ruts in the driving lane had a distinctive "dual-wheel" trace (see photographs 1 and 2). Figure 2 shows the rutting profile of the southbound direction for both the driving lane and the passing lane between MP 235 and MP 242 at the end of the 3-1/2 years of service (7).

The surface texture appeared normal for a pavement of this age. There had been no signs of flushing or bleeding within the project limits. To visually inspect each layer, a 4 to 5-inch wide slice of the southbound driving lane (12 feet wide) was removed. Photographs 3 and 4 show the pavement slice which was extracted at MP 237.05. From the removed section, the various paving layers could easily be identified and it was possible to see how rutting had varied as a function of depth (7).

The thickness of the pavement at the shoulder stripe or at center lane was taken as a datum (Zero rutting). The following table shows the thickness of each layer at center lane and at the wheel path and their corresponding reduction in thickness.

Table 1
Measured Rutting as a Function of Depth

Mat	Layer Thickness		Reduction in Thickness	
	Center	Wheelpath	Inches	Percent
Top	1.60"	1.35"	0.25"	16
Middle	1.70"	1.60"	0.10"	6
Bottom	1.65"	1.55"	0.10"	6
Total	4.95"	4.50"	0.45"	9

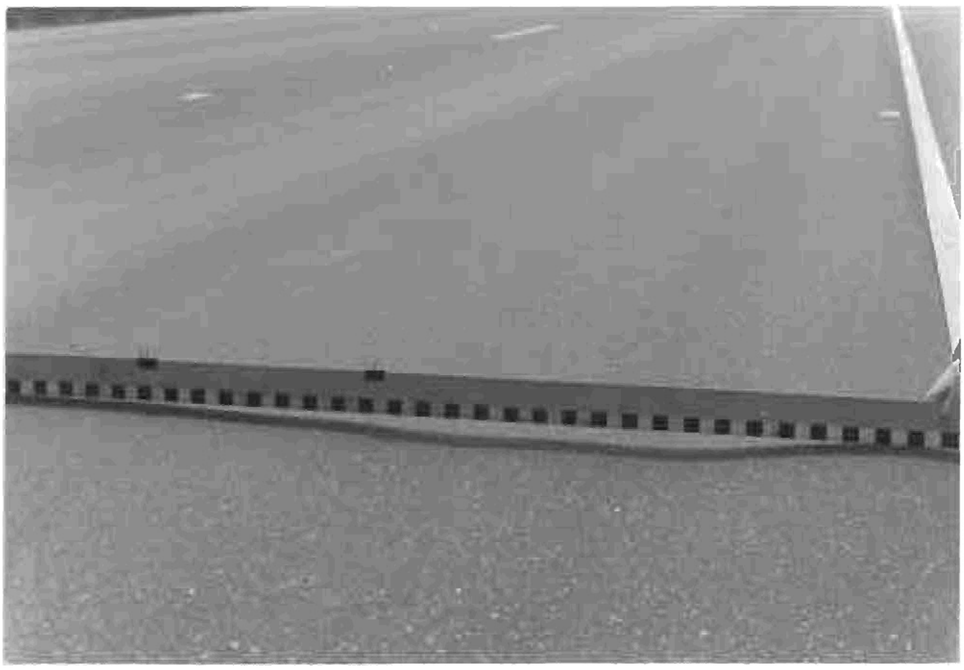


Photo 1 & 2: I-25 Southbound with dual-wheel trace ruts approaching one inch.

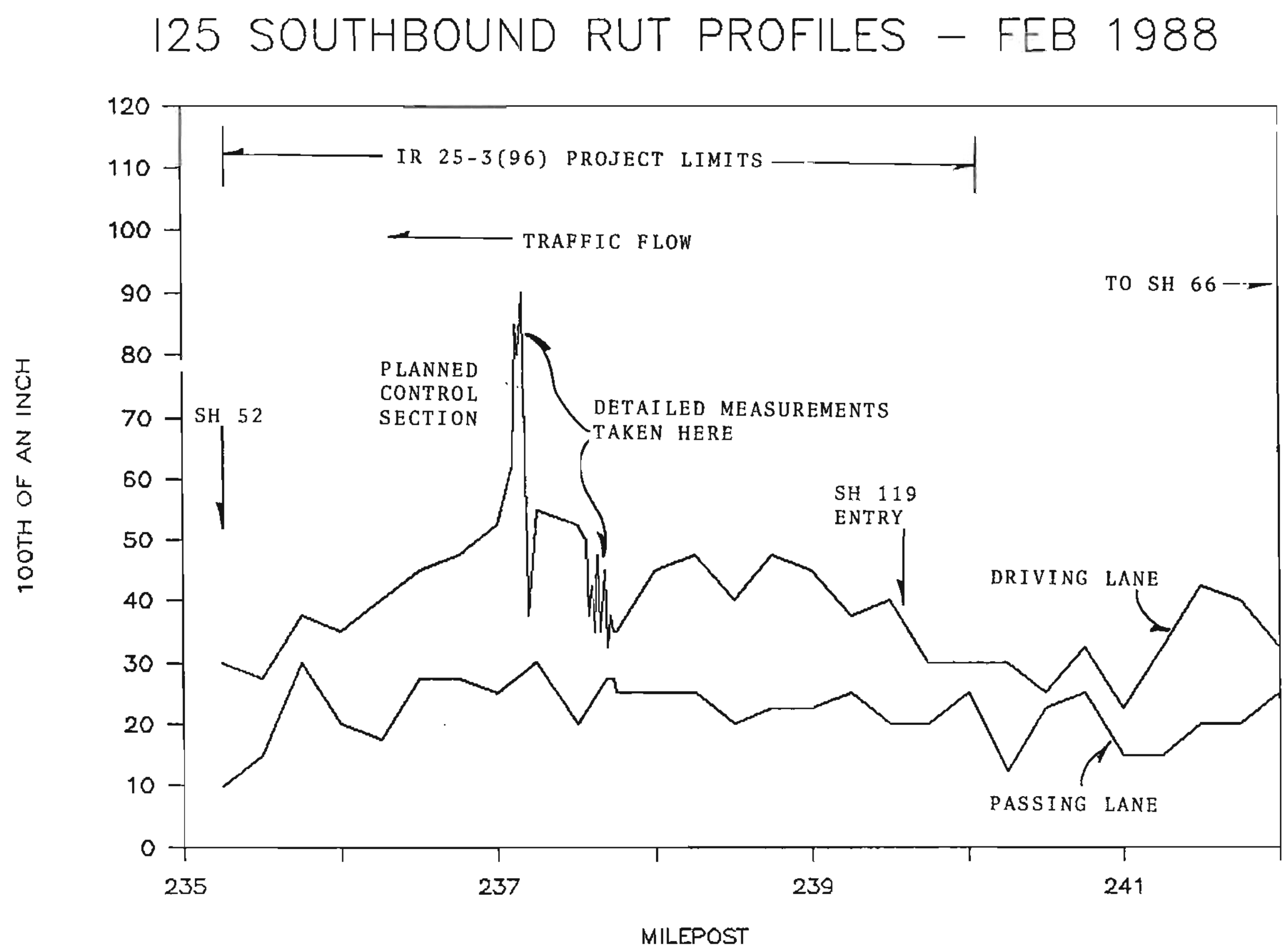

I25 SOUTHBOUND RUT PROFILES – FEB 1988
IR 25-3(96) PROJECT LIMITS
TRAFFIC FLOW
TO SH 66
PLANNED CONTROL SECTION
DETAILED MEASUREMENTS TAKEN HERE
SH 52
SH 119 ENTRY
DRIVING LANE
PASSING LANE
100TH OF AN INCH
MILEPOST
0
10
20
30
40
50
60
70
80
90
100
110
120
235
237
239
241

Photo 3: Extracted sample - I-25, Southbound MP 237.05

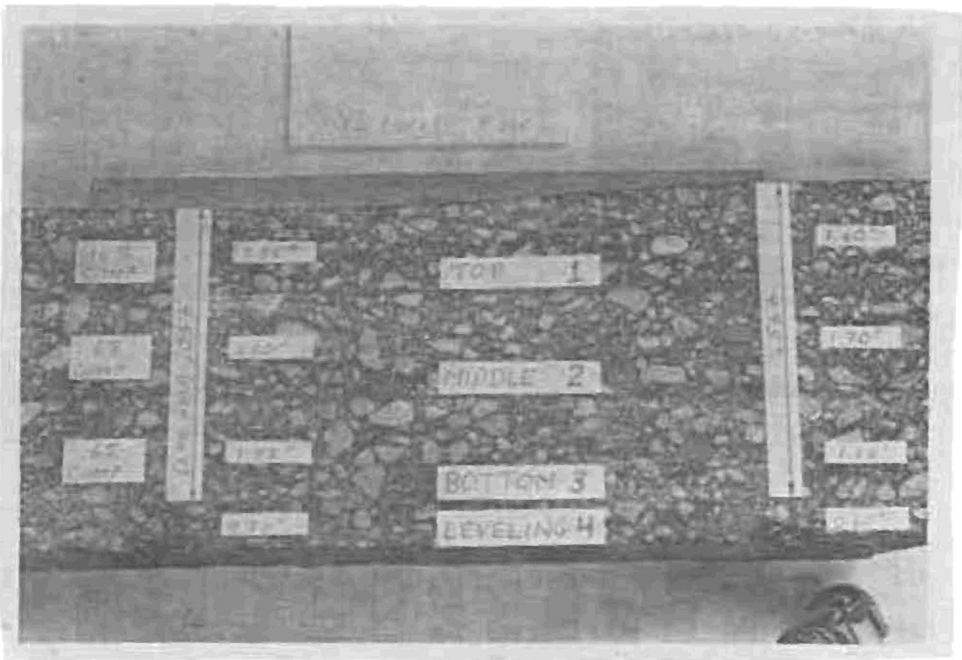


Photo 4: Measurements showed that the largest part of the rut developed in the top mat of the pavement.

As shown in the above table, the top layer exhibited the greatest rutting (16 percent), while the middle and the lower layers had been consolidated only about 6 percent. This suggests that rutting is indeed a surface phenomenon.

A review of the laboratory tests conducted on the extracted cores showed that the material used for the project met the required design specification. The stability achieved was 37.8, the asphalt content was 5.89 percent, and density was 97.5 percent. However, no conclusive correlations were found between the measured rutting and the laboratory data such as void, stability, resilient modulus, specific gravity, etc. The concept being investigated was an attempt to identify what differences existed between pavement characteristics of the wheel path and the pavement in the center and at the pavement edge.

Truck-axle loads remain the prime suspect in contributing to the rutting problem at this location.

C. Construction

During the summer of 1988, five miles of the southbound lanes between MP 235 and MP 240 which contained the worst rutting were rehabilitated. The rehabilitation plan called for milling out two inches of the driving lanes and replacing it with modified hot bituminous pavement (HBP Gr E) that had a high modulus (475,000), higher voids (4-8), and a required Hveem of 40. This "super-stable" mix replaced the worst part of the rutted pavement. In order to protect the modified HBP, the entire width of the road was then covered with a standard 3/4 inch Plant-mixed seal coat (PMSC Type B). The PMSC also covered the 1/4 inch ruts that had developed in the passing lane. Typical sections are shown in Figure 3 and Figure 4.

FIGURE 3
Project IR 25-3(96)
EXPERIMENTAL TEST SECTION

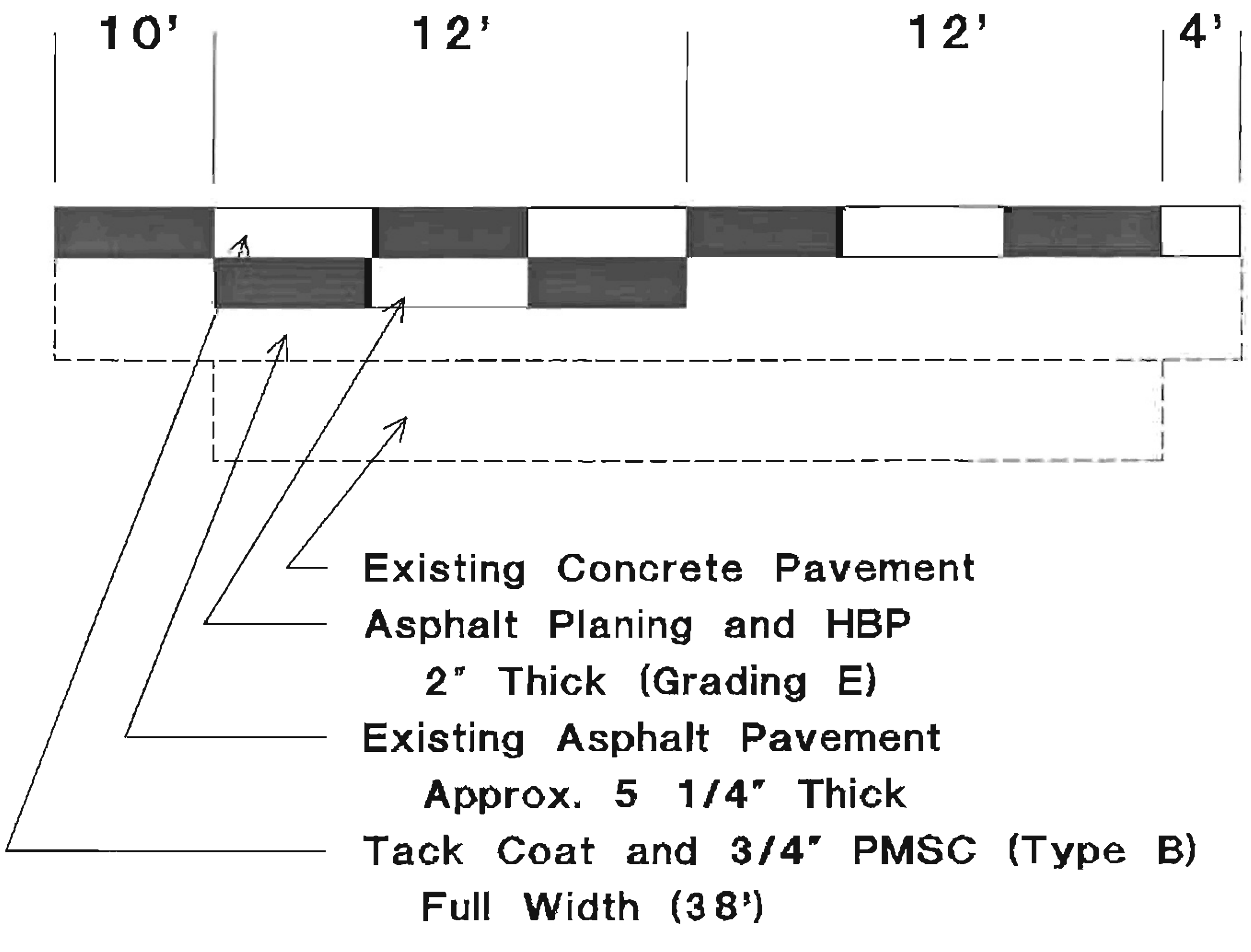
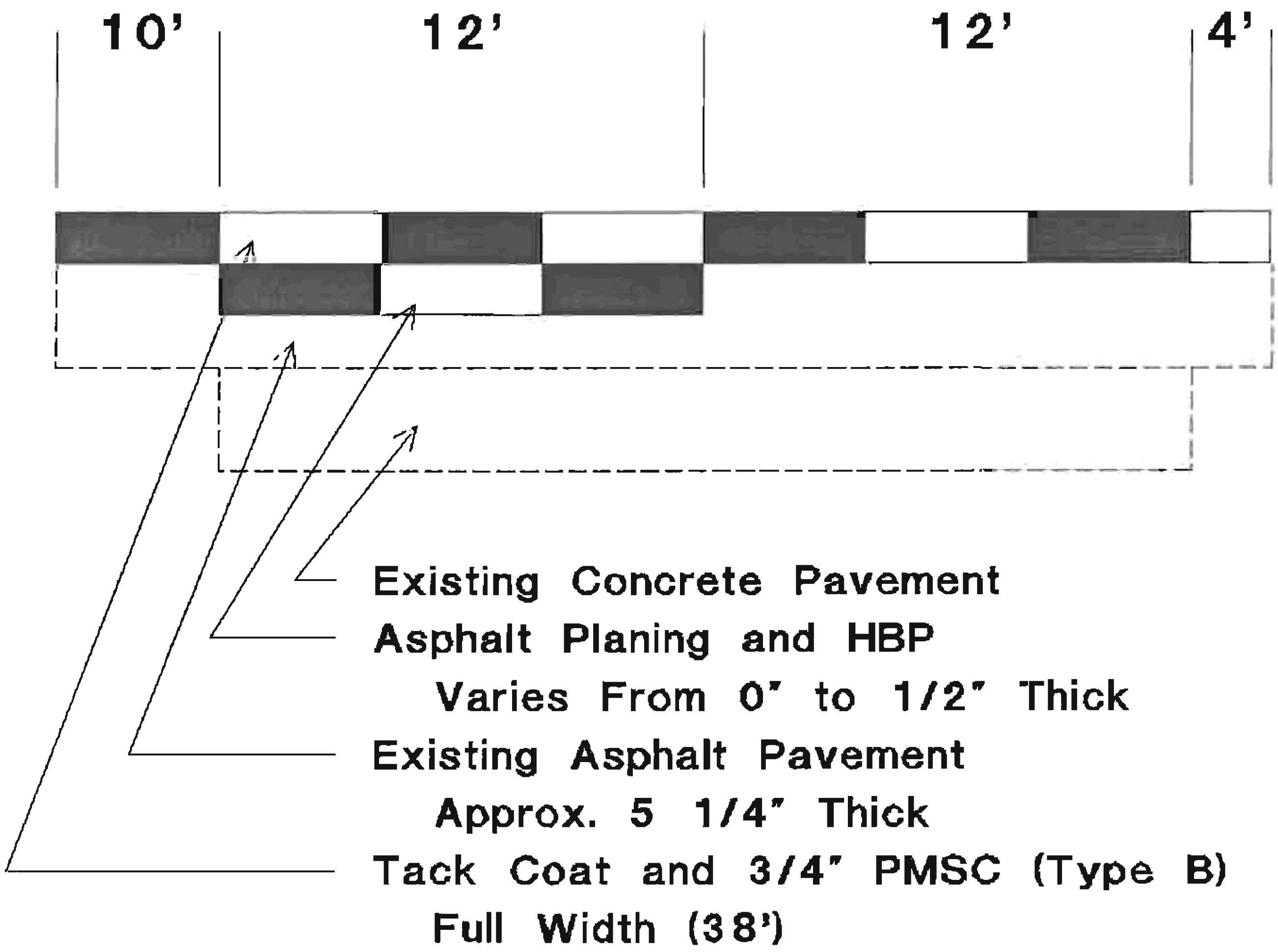


FIGURE 4
Project IR 25-3(96)
EXPERIMENTAL TEST SECTION



The project specification did not require the use of 100 percent crushed material for the modified HBP. The Contractor, Siegrist, used a mixture of natural and crushed material for the fines in the mix and had no trouble meeting the material requirements for the gradations.

The roto-miller moved in the direction of the traffic with an average rate of 14 feet per minute. The mill tailings were effectively cleaned with several mechanical brooms and a loader (Photographs 5 and 6). The paving operation followed several hours behind. After all the milling and HBP placement had taken place, the contractor returned to the beginning of the project (MP 240) and placed plant-mixed seal coat (PMSC) on the passing lane and inside shoulder. Next, the driving lane and then the right shoulder was paved with PMSC. All three passes of PMSC were completed in three days.

A test section and a control section were established to monitor and evaluate the performance of the newly-paved overlay in the southbound lanes (Figure 5). In addition, a control section was also established to monitor the progress of rutting in the northbound lanes. The following is a brief description of the control and test sections:

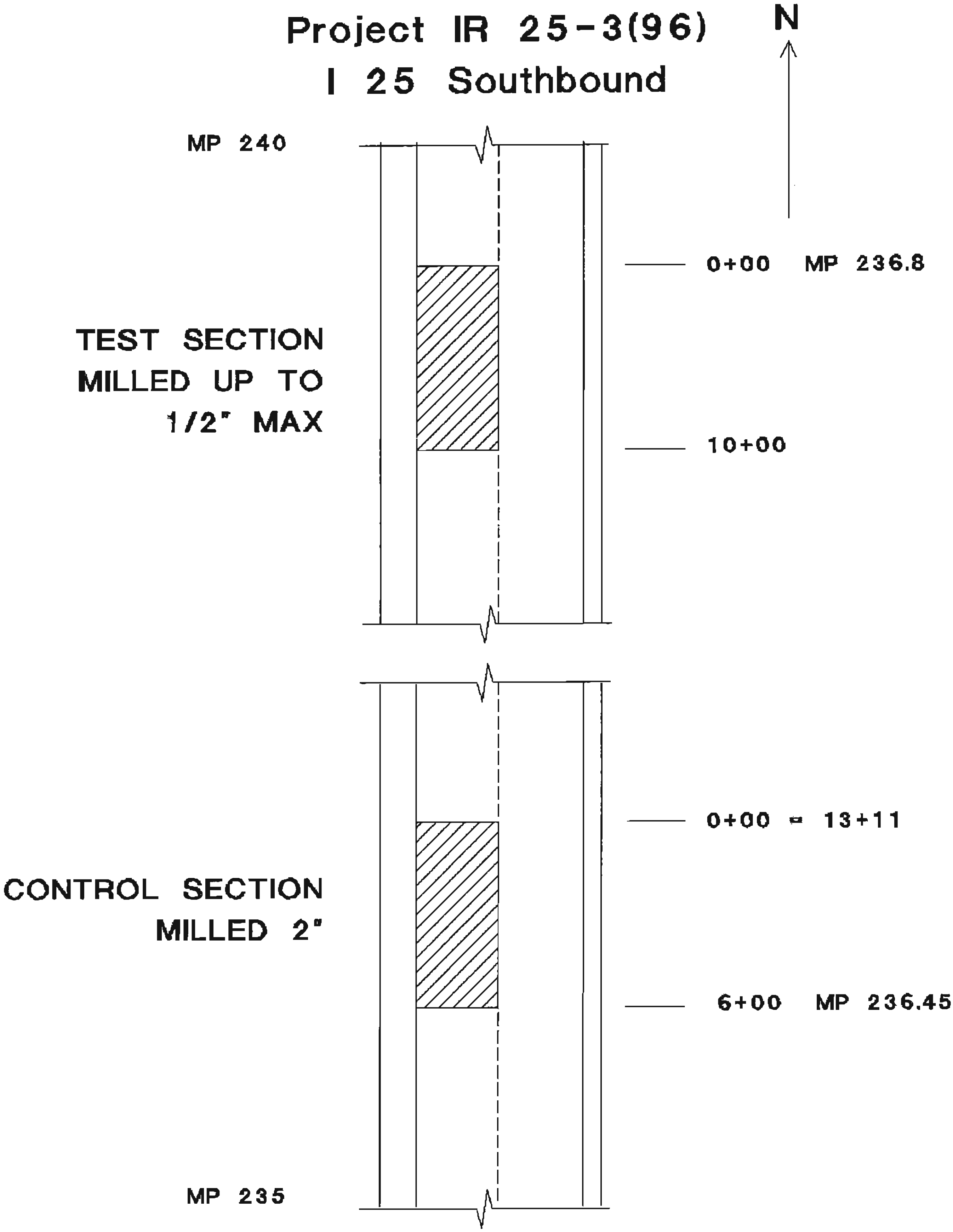
1- The southbound test section was milled 1/2 inches and overlaid with 1-1/4 inch of PMSC.

2- The southbound control section was milled out 2 inches and overlaid with 2 inches of high stability asphalt and 3/4 inches of PMSC.

3- The northbound control section is the original 5-inch overlay with no rehabilitation.

Photo 5 & 6: View of the milling operation from behind and side.

Figure 5
Project IR 25-3(96)
I 25 Southbound



D. Construction problems

One of the major problems encountered was with the AC content of the PMSC. The AC content of the PMSC used exceeded the amount design called for; as a result the entire length of the driving and the passing lanes started to flush (Photographs 7 and 8). The PMSC was rejected and the contractor was directed to mill out the defected material and replac it with a PMSC that contained the proper amount of asphalt cement.

Among other minor problems was the clogging up the milling machine on the warmers days which slowed the milling operation. This problem was alleviated by pre-wetting the pavement surface with a water truck. Traffic delays were also more than expected.

V. Data Acquisition and Analysis

Rutting Data

Rutting progression was used as the primary method of evaluating and monitoring the performance of the rehabilitated southbound lanes. A complete history of rut profile taken between 1988 and 1992 for the southbound lane, and between 1984 and 1992 for the northbound lanes are shown in figures 6 through figure 17. These rutting profiles were taken in the driving lane and in the passing lane for both the left-wheel-path and the right-wheel-path (DLRWP, and DLLWP, PLRWP, and PLLWP).

An analysis of the rutting measurements taken to date revealed no appreciable rutting for the southbound lanes during the first year after the rehabilitation (1989). The maximum rut depth measured 0.20 inches in the right-wheel-path at station 7+00 in the test section. The rut depth for the southbound control section during the first year averaged less than 0.1 inches. The

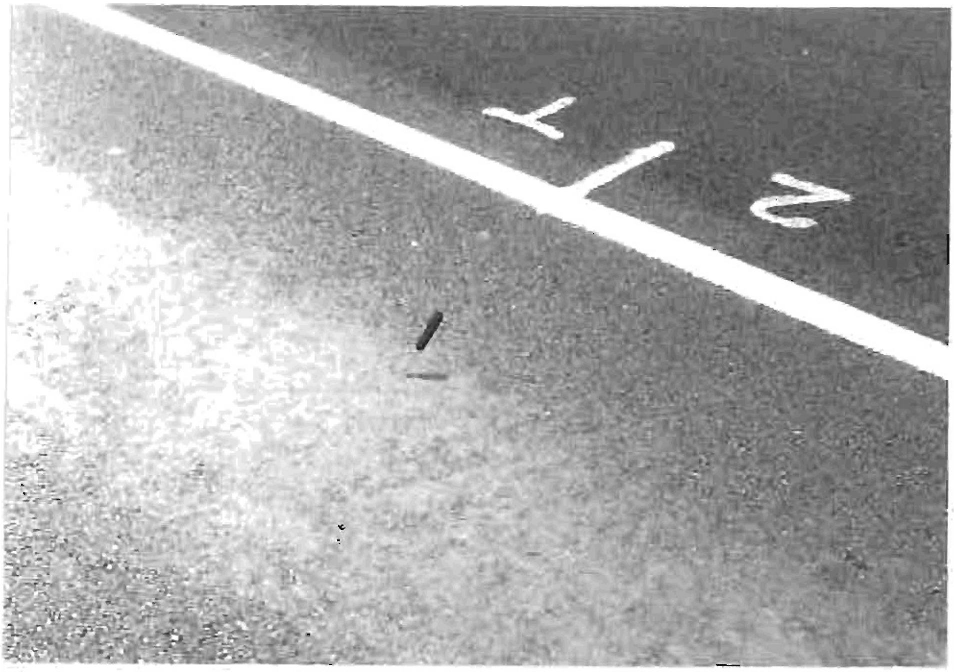


Photo 7: Flushing of the PMSC at MP 236.6, note the difference in texture between the shoulder and the driving lane.

Photo 8: Flushing of the PMSC was noticeably higher around MP 240.

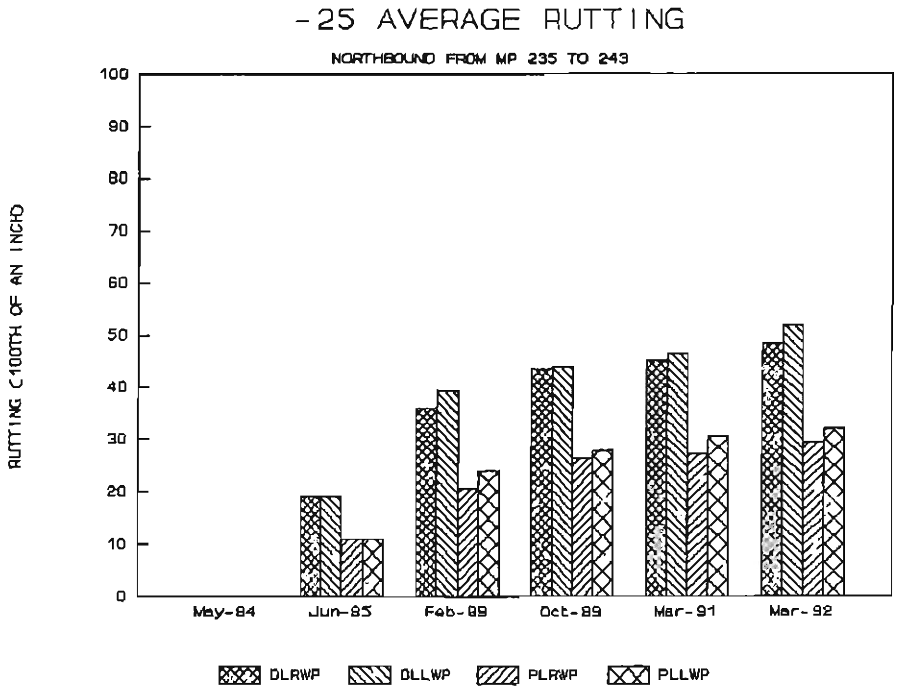


Figure 6

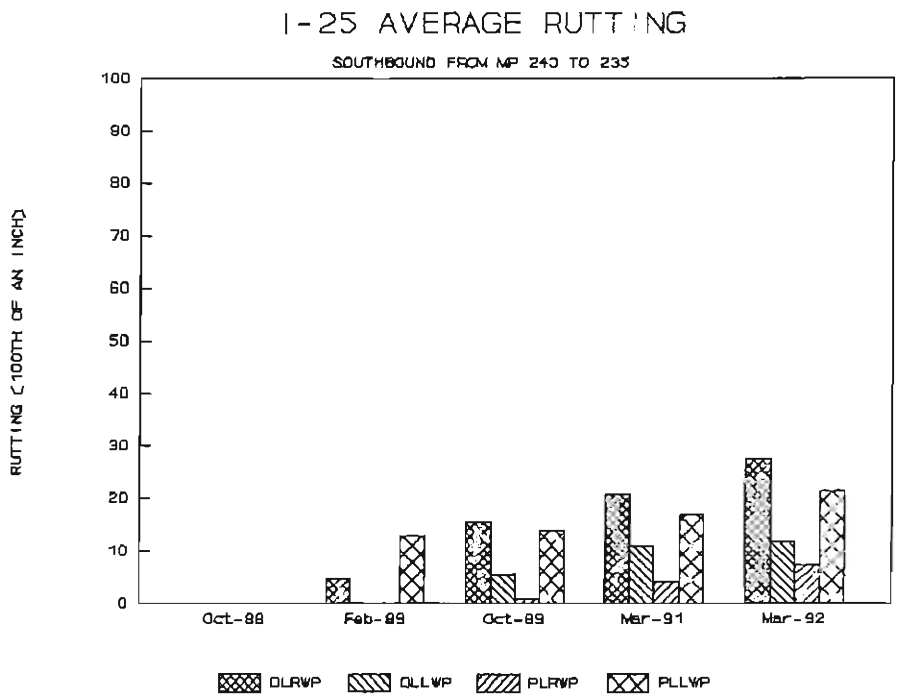


Figure 7

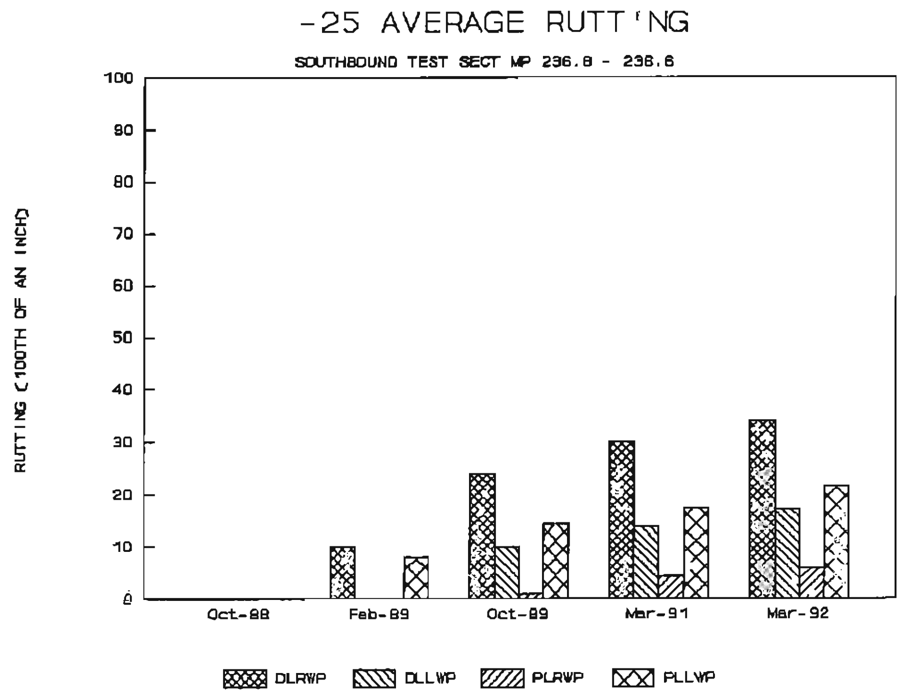


Figure 8

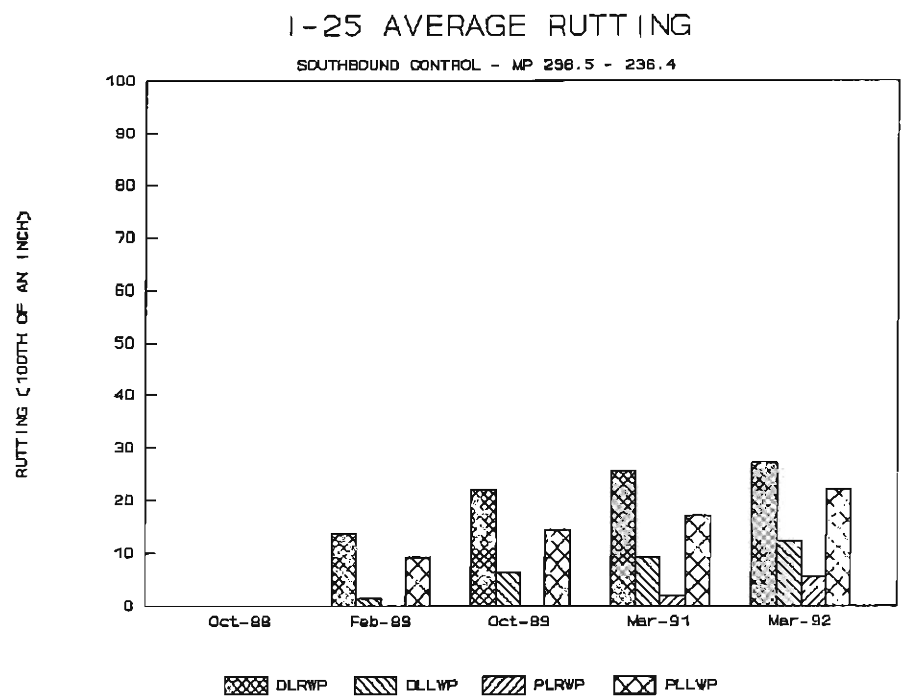


Figure 9

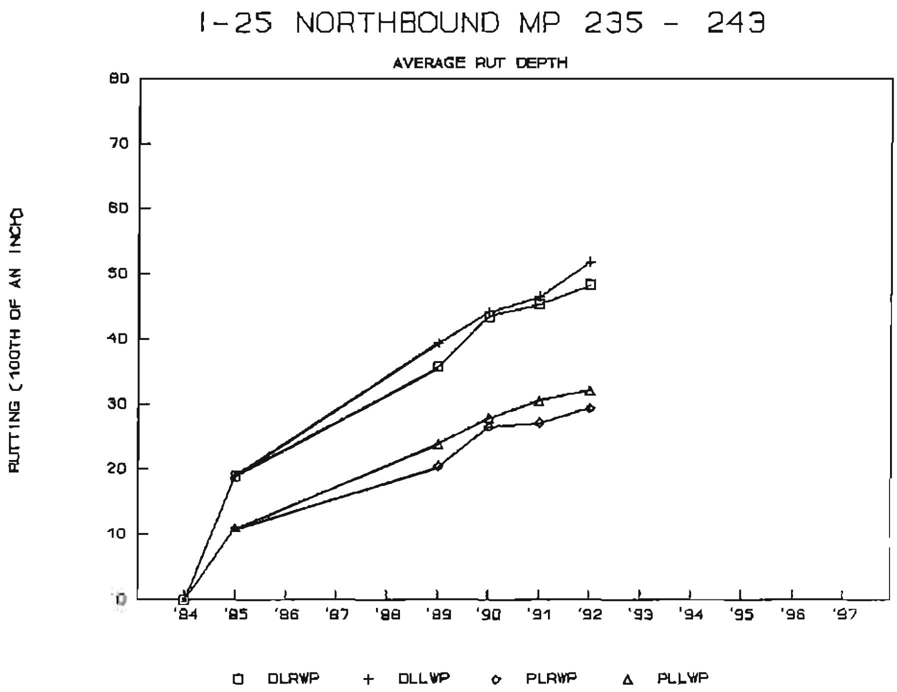


Figure 10

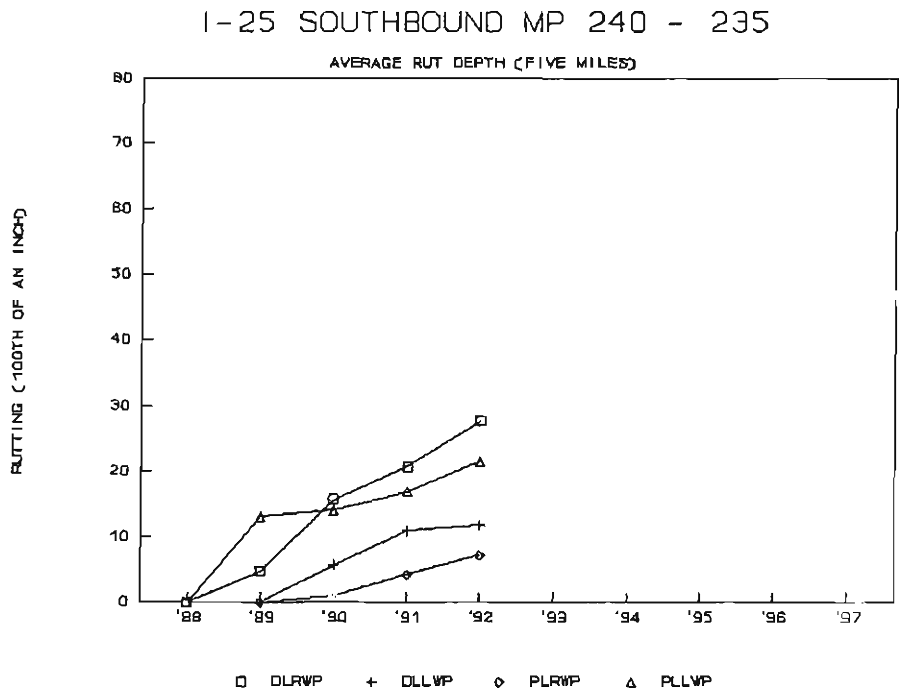


Figure 11

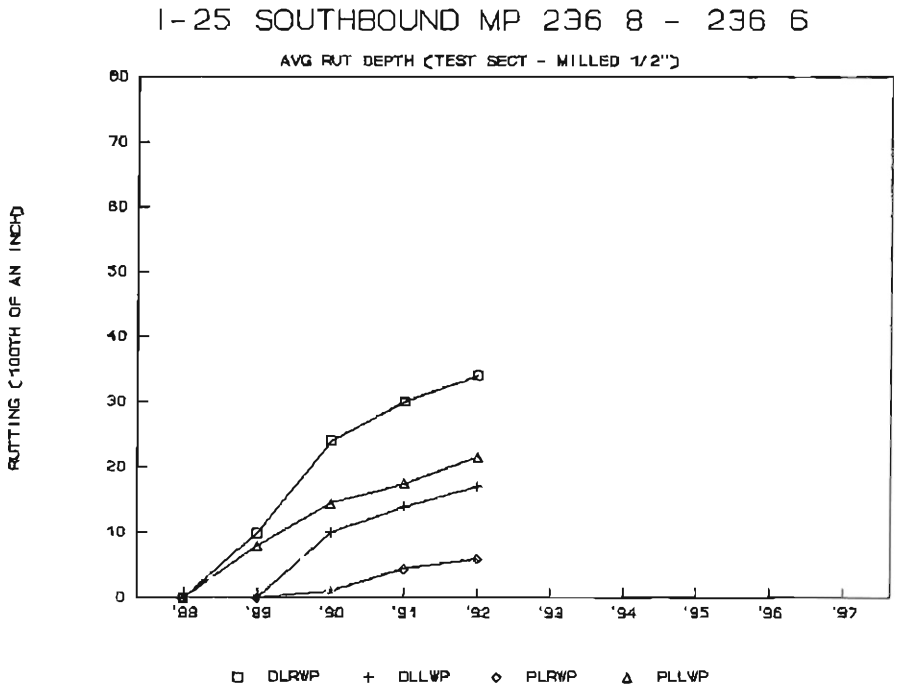


Figure 12

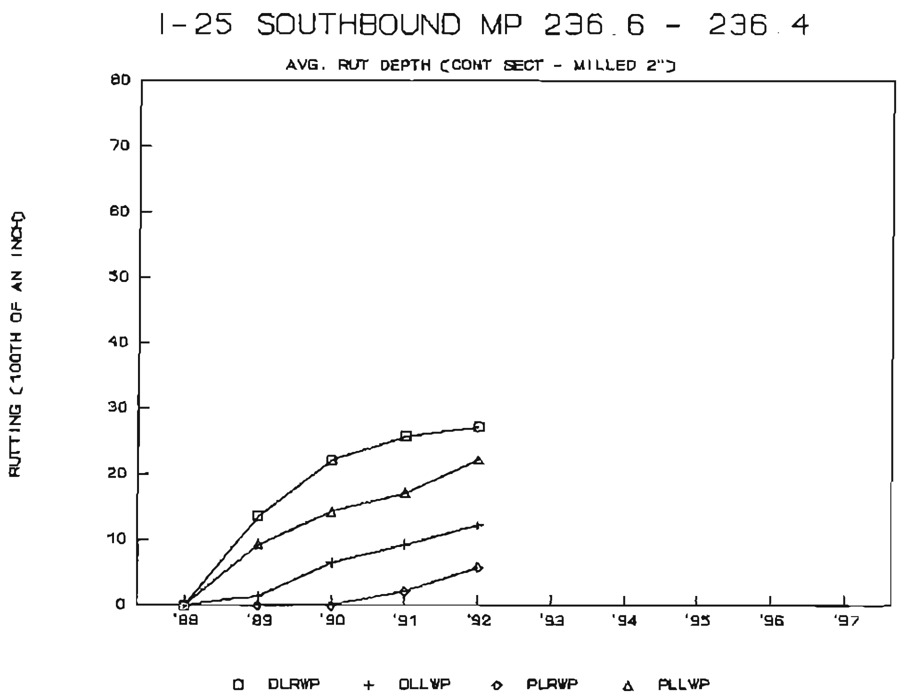


Figure 13

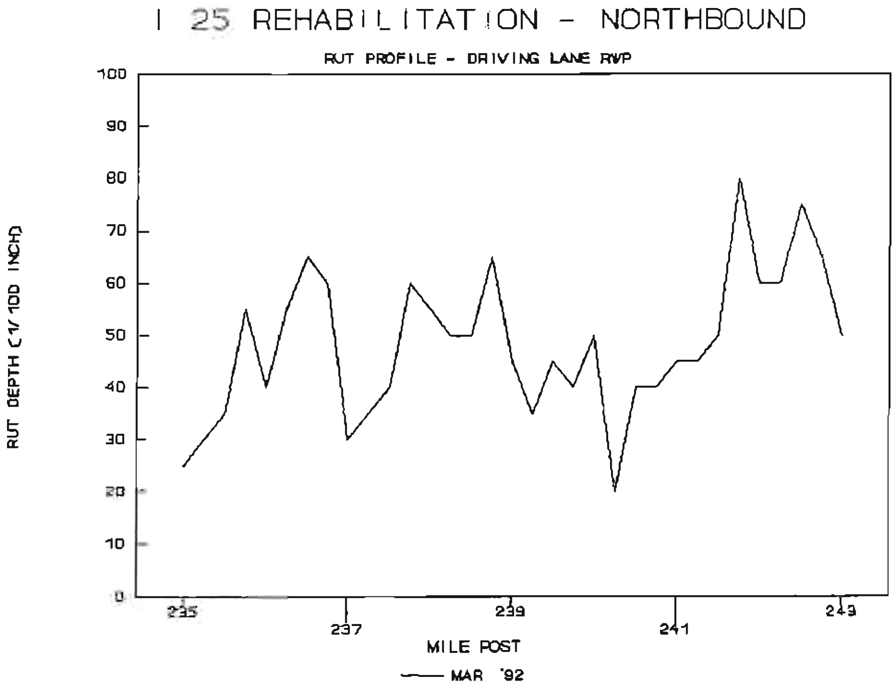


Figure 14

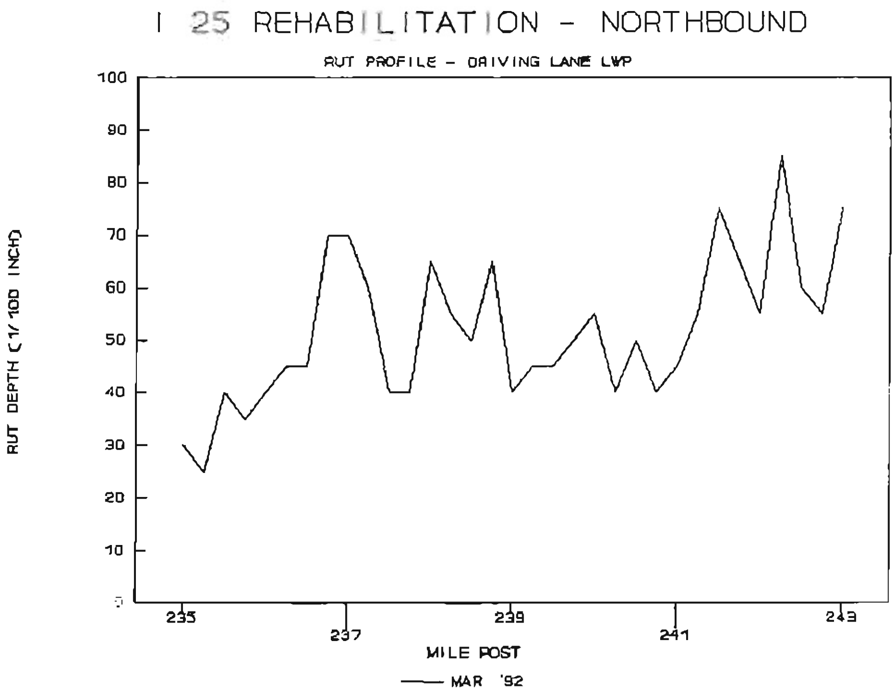


Figure 15

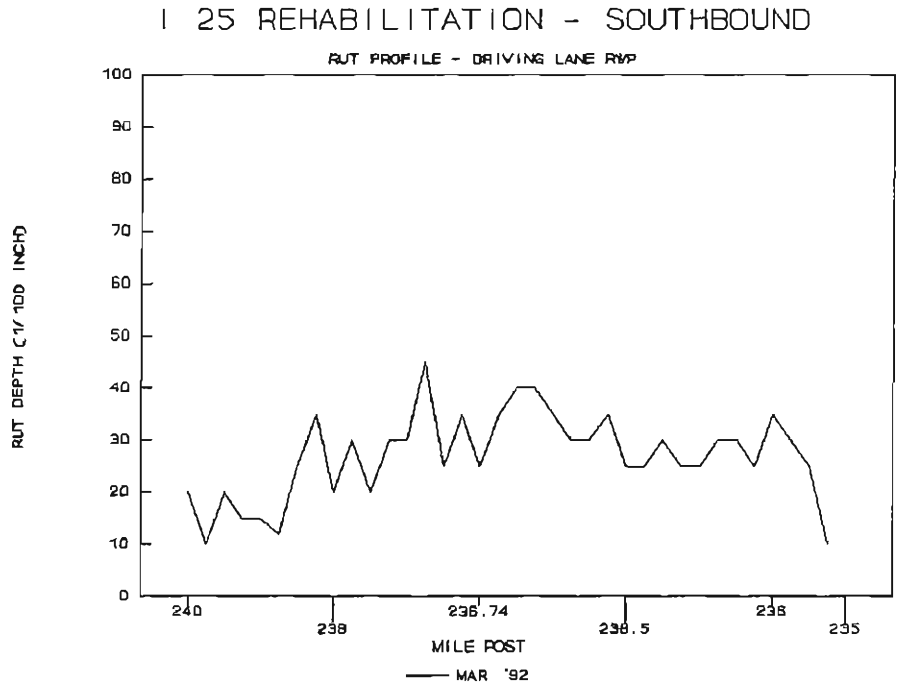


Figure 16

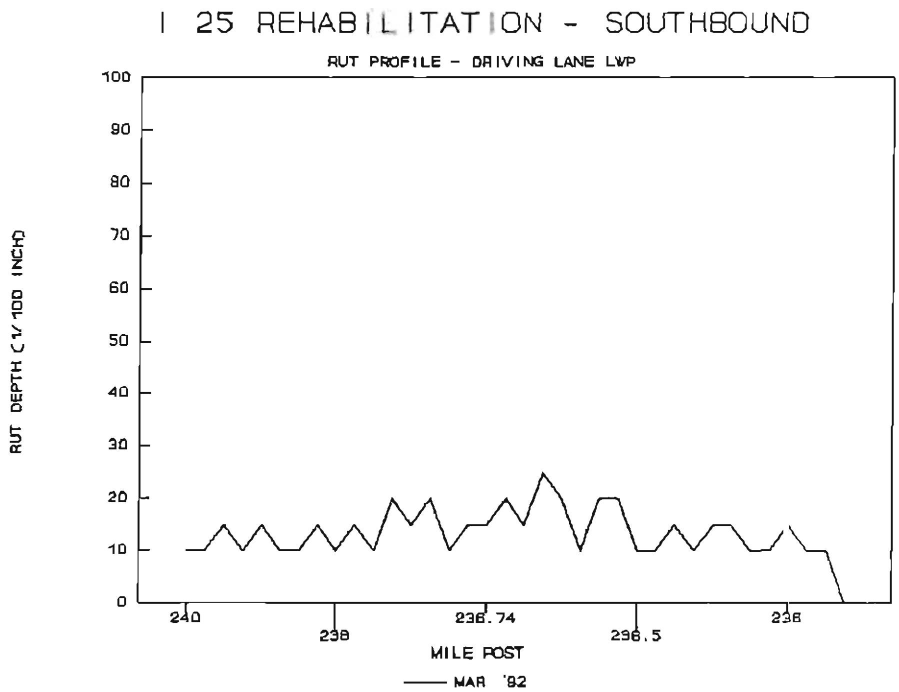


Figure 17

average rut depth for the northbound lanes measured 0.40 inches during the 1989 season. However, occasional rut depth of up to 0.60 inches and many above 0.45 inches were detected.

A review of the rut profile taken during the fall of 1990 through 1992 indicated a progressive increase in the magnitude of rutting for both the southbound and northbound lanes. The increase in the rate of rutting was more pronounced for the southbound test section than other sections. The magnitude of maximum rutting in the northbound driving lane measured approximately 0.90 inches and approximately 0.5 inches for the southbound lane.

The following is the summary of the findings after 4 years of rut depth monitoring:

- Rutting is gradually but surely returning to the rehabilitated southbound lanes.

- Rutting is recurring faster in the test section (with minimal milling) than where the pavement was milled two inches and replaced with high-stability mix (control section).

- The early southbound rutting is largely in the outside wheel paths.

- Rutting on the unrehabilitated northbound lanes is continuing at a rate of 0.06 inches per year and now averages 0.5 inches with measured ruts up to 0.9 inches in one area.

Assuming that an average rut depth of 0.50 inches is the limit that calls for rehabilitation, then the northbound lanes should be rehabilitated during the 1993 construction season. This indicates a nine-year life expectancy for the I-25 northbound lanes.

The southbound control section, which was milled to a depth of two inches, repaved with HBP Gr E and overlaid with 3/4 of an inch PMSC will reach the average rut depth of 0.50 inches in 1977. This again is a life span of nine years.

Projections for the southbound test section indicates a life span of 6 years. This means that the southbound test section will average a rut depth of 0.50 inches in 1994. The test section was milled 1/2 inches and overlaid with 1.25 inches of PMSC.

Appendix B shows a complete survey of the rut-depth measurements taken from 1988 to 1992.

Cracking Data

Only the test and the control sections in the southbound driving and passing lanes were monitored for cracking. There are appreciable amounts of cracks in both the test and the control sections. However, these cracks are primarily in the form of transverse reflective cracking. No fatigue cracking (load-related cracks) was detected during the four years of investigation. The load-related cracks are usually initiated at the bottom of the pavement layer where tensile strains are highest. Repeated traffic loads then propagate the cracks to the surface, forming alligator cracking.

The amount of reflective cracking developed in the test section measured 54.8 feet/station/roadway width and in the control section measured 68.3 feet/station/roadway width.

Cores

Two sets of cores were taken from the control and the test section in a grid system in the southbound driving lane (Photograph 9). Each set consisted of 4 cores in the right-wheel-path (RWP) and 4 cores in the center of the lane. These cores were tested for density, % A.C., gradations, stability, % void, resilient modulus, and lottman.

The laboratory tests were largely inconclusive; however, bulk specific gravity of the top layers was consistently higher than the bulk specific gravity of the bottom layer by about 3.5 percent. This suggests that the rate of vertical deformation for the top layer is higher than the bottom layer.

VI. Conclusions and Recommendations

Based on the findings of this study and the literature reviewed the following conclusions and recommendations are presented:

High-stability asphalt is not resistant to rutting, and rutting is returning to the rehabilitated southbound lanes, but of a slower rate than the standard mix.

Rutting is recurring faster in the test section (with minimal milling) than where the pavement was milled two inches and replaced with high-stability mix (control section).

Rutting on the unrehabilitated northbound lanes is continuing at a rate of 0.06 inches per year and now averages 0.5 inches with measured ruts up to 0.9 inches in one area

Assuming that an average rut depth of 0.50 inches is the limit that calls for rehabilitation, the southbound

Photo 9: View of the cores taken from the southbound lanes.

control section will have a life span of nine years while the southbound test section will have a life span of 6 years.

A pre-construction examination of the existing pavement indicated that the top mat makes the greatest contribution to the total rutting.

The results of laboratory tests were largely inconclusive. No correlations were found between the measured rutting and pavement parameters such as stability, asphalt content, gradations, voids, etc.

Even though stripping was not a problem for this project, similarly designed pavements in some parts of Colorado showed stripping to be the primary mode of failure. This phenomenon has been discussed in details in reference 8, " Rut-Resistant Composite Pavement Design". Of the ten sites constructed with high-stability designed mix, three were distressed due to stripping.

Recommendation For Further Research

A number of recent studies suggest a definite correlation between rutting and material passing No. 200 sieve. Reference 5, "Investigation of Premature Distress in Asphalt Overlays on IH-70 in Colorado", recommends for Colorado DOT to consider limiting the amount of P200 in mixtures used on higher volume pavements.

VII. Implementation

The results of this study demonstrated that high-stability pavement topped with a plant-mixed seal coat is not significantly better than the standard mix. In general, the high-stability mix could not eliminate the formation of rutting; however, it was able to slow down the rate of rutting.

CDOT should limit the amount of P200 in mixtures used on higher volume pavements.

CDOT's participation in the SHRP performance-based asphalt binder and mixture specification, and inclusion of innovations in asphalt pavements such as Stone Matrix Asphalt (SMA), is greatly recommended.

VIII. **References**

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Appendix A

THIRTEENTH ANNUAL ASPHALT PAVING SEMINAR

"REHABILITATION OF CONCRETE FREEWAYS WITH ASPHALT"

KENNETH L. WOOD
COLORADO DEPARTMENT OF HIGHWAYS
DECEMBER 5, 1985

REHABILITATION OF CONCRETE FREEWAYS WITH ASPHALT

The purpose of this paper is to describe the design, construction and performance to date of an 8 mile long rehabilitation project on a portion of I-25 using a 5 inch asphalt overlay.

The project is located on I-25 between SH 52 on the south and SH 66 on the north. Traffic consists of 29,8000 ADT with approximately 15% trucks. The adjacent land areas are typically irrigated and dryland farms. The interstate runs parallel to the front range of the Rockies, which are approximately 20 miles to the west.

The existing surface was a 25 year old Portland Cement Concrete Pavement (PCCP), 8 inches thick, built with 4 foot and 10 foot asphalt shoulders, 3 inches thick. Underlying the old PCCP is an untreated aggregate base.

DESIGN

The project consisted primarily of placing a 5 inch thick asphalt pavement overlay. The design procedure used was taken from the Asphalt Institutes Information Series No. 177, entitled, "Asphalt Overlays for Heavily-Trafficked PCC Pavements," published in February, 1981.

The first step in the overlay design procedure is to determine the pavement condition. This includes deflection measurements, identifying distresses such as longitudinal cracking, transverse cracking, rutting, depressions, crack spalling, faulting, corner breaks, reactive aggregate, and the severity or extent of each.

Deflection testing determines the response of the pavement for a given load, which was a single axle test load of 18,000 lbs. Deflection measurements were made using a Benkelman Beam. The deflections measured should reflect the most severe environmental circumstances, which are usually found in the spring of the year. If taken during other seasons, adjustments are recommended.

Deflection measurements were taken at the edges of transverse joints, and transverse cracks both on the approach and leave sides. They were taken at approximately two hundred foot intervals.

Some care should be exercised when using the Benkleman Beam. For example, the best results were obtained when air and slab temperatures were less than 50°F. Otherwise, a correction should be made to compensate for partial slab lock-up. At 90°F. it was found that there was complete slab lock-up, making it impossible to measure any deflection.

Using the Benkelman Beam required that several measurements be taken for averaging to obtain a representative value because it apparently is so variable. Analyzing individual PCCP slabs for such things as voids is not recommended. It should be noted, however, that the surface distress survey correlated to the deflections measured for the various AREAS of the project.

The other portion of the condition evaluation or survey is identification of surface distress and severity. The technique used for this is a walk through of the project mapping cracks and making notes concerning the conditions found, such as transverse cracking, longitudinal cracking, rutting, faulting, and reactive aggregates.

Overlay Selection

The Hot Bituminous Pavement overlay thickness is a function of the PCCP slab lengths and the differences in the normal maximum and minimum temperatures in the area of the project over a period of one year. For this project the slab lengths were 20 feet and the temperature differential was 71°F. This results in a 5 inch thick overlay which is read from a chart in the Asphalt Institutes design procedure.

This thickness is intended to control the horizontal tensile strain. The longer the slab length the more horizontal expansion/contraction and consequently the more thickness required. Also, higher temperature differentials require more thickness. Next, the vertical deflections measured on the project must be checked against a maximum allowable deflection of 0.014 inches and a maximum allowable differential deflection at the transverse joints and cracks of 0.002 inches. Although the horizontal movements and vertical deflections both contribute to the same crack because they share the same plane of weakness, it is the differential vertical deflections that are critical. The loading rate of these deflections are more than 1000 times faster than those developed by the horizontal contraction/expansion. Therefore, the maximum allowable deflections previously mentioned must be satisfied as shown in the following example.

EXAMPLE OF DESIGN CALCULATION:

Mean Vertical Deflection (Measured on project)	=	.016"
Mean Load Transfer Ratio (Measured on project as the ratio of the deflection on one side of the joint or crack to the deflection on the opposite side)	=	1:1.2
Slab Lengths	=	20 feet
Area Temperature Differentials	=	71°F.

FROM ASPHALT INSTITUTE DESIGN CHART--entering temperature differential (71°F.) and slab lengths (20 feet) the design thickness is 5 inches.

CHECK VERTICAL DEFLECTION--(deflections are reduced at the rate of 5% per inch.)

5% x 5" = 25% reduction of deflection

.25 x .016" = .004"

.016" x .014" = .012" (which is less than the .014" allowed)

CHECK DIFFERENTIAL DEFLECTIONS AT JOINTS--(a mean load transfer ratio of 1:1.2 represents a difference of 20% in deflection from one side to the other.)

.012 x 0.20 = .002" (which is equal to the .002" allowed.)

The resultant 5 inch overlay satisfies the horizontal movements plus the vertical and differential deflections at the joints and cracks.

In addition to this, it was decided to remove and replace severely cracked and spalled slabs. Approximately 1% of the slabs were removed and replaced in the driving lane. None in the passing lane. The intent here was to remove those slabs that were broken and in several pieces, which may result in early reflective cracking due to excessive movement. The condition survey was of great value in determining the removals.

CONSTRUCTION

The overlay was placed in four lifts, from the bottom to the top they were 3/4", 1 1/2", 1 3/4", and 1 1/2".

Before the overlay was placed, the slabs that were severely cracked and spalling at the joints and cracks were removed and replaced. Removal was done with as little disturbance as possible to the underlying aggregate base course. The slabs were diamond sawcut full depth before removal. This technique also results in a minimum of disturbance to the adjacent concrete during removal.

Load transfer was reestablished in the transverse joints at the slab replacement locations. This was accomplished by placing smooth dowels at the interface of the old and new slabs. Six were placed at each end, three in each wheel path. The intent is to eliminate as much of the differential movement at the joints as possible.

One of the big advantages of rehabilitation with an asphalt overlay, on this project, was that traffic could be moved over to the adjacent lane and maintained during overlay construction even with a 30,000 ADT. This was also true of the slab removal and replacement technique. The old PCCP slab that was to be replaced was removed in the morning, poured that afternoon and opened to traffic by 5-6 p.m. One lane of traffic was always maintained during construction and both lanes overnight.

Using the Hveem method for design, the bottom 2 lifts of the overlay consisted of a minus 3/4" material, which is a Grading E. The asphalt content was established at 5.8% The voids at this content were 3.7% and the stability value was 38. The top two lifts placed on the project were a minus 1/2 inch material meeting the requirements for Hot Bituminous Pavement, Grading EX. The asphalt content was again established at 5.8%. The voids at this content were 4.8% and the stability value was 38. A stability value of 37 was required for this project. This reduced the deflections at the rate of 7 1/2% per inch of overlay, resulting in a total theoretical reduction of 37 1/2% (5" x 7 1/2% per inch).

The overlay started in September of 1983. Weather allowed the contractor to work until late fall which was the middle of November. All but a small portion of the project had 3 of the 4 required total lifts placed before the "paving season" ended in November. A small portion of the south end of the southbound lane had only 2 lifts placed. As a result, the incomplete project was open to traffic the following winter. In the spring (1984) the overlay was started and then it was finished that summer.

During the completion of the overlay that summer (1984) it was decided to extend the overlay, which was 5" and 4 lifts, to the north to accommodate an approach to an adjacent rehabilitation project. The entire thickness was placed that summer. This is important to the Department because there was much concern about the fact that all 4 lifts were not able to be placed, the previous year, before the paving season ended. There is now an opportunity to evaluate the performance of the two areas: 1) one area where the full 5" thickness was placed in one season, and 2) other areas where only 2 and 3 lifts were placed in one paving season.

PERFORMANCE

The Research Branch of the Department has set up test and evaluation sections on the project. These sections have been located in the area where all 4 lifts were placed during the summer of 1984 and also in the areas where only 2 and 3 lifts of the asphalt overlay were placed in the paving season of 1983 before winter stopped placement of the pavement.

Concerns were expressed over not being able to place all 4 lifts of the pavement overlay before winter because of the effect it may have on the long term performance. Some felt the late fall, winter and early spring traffic and environmental conditions would result in early pavement distress. Since the full design thickness was not placed, the strength to resist the horizontal and vertical movements of the underlying 20 foot slabs would not be available. Remember that as far as the vertical movements were concerned, deflections are reduced at the rate of 7 1/2% per inch. Not being able to complete the top 1 1/2 inch lift allowed 11% more deflection (1 1/2" x 7 1/2% = 11%) and more for the small portion of the southbound lane where 2 lifts were placed.

Approximately 1500 feet of the southbound lane where the 2 and 3 lift sections were placed prior to the end of the paving season has been established as a test evaluation section. Another test and evaluation section was established at the north end of the project where it was extended and the full 5" was placed in a single paving season.

Along with evaluating the specific performance data collected on this project it is also the Research Branch's intention to compare the performance of this project to the adjacent projects north, which are thick unbounded PCCP overlays. Test and evaluation sections have been established on them also.

PERFORMANCE DATA

The following data is the result of two evaluations to date, the first in April, 1984 and the second in June, 1985. Since the extended approach (placed full depth in one season) was not completed in April, 1984 there is no data for the first evaluation, nor is there April, 1984 data for smoothness, rutting, bleeding and deflections.

Cracking

	*April, 1984	June, 1985
3 Lift Section	228	242
2 Lift Section	0	118
Full Thickness Section	---	0

*Overlay not complete

Smoothness (Profilograph)

	June, 1985
3 Lift Section	
Driving Lane	2 inch/mile
Passing Lane	2.3 inch/mile
2 Lift Section	
Driving Lane	3 inch/mile
Passing Lane	1 inch/mile
Full Depth Section	*.1 inch/mile

*Some portions 0.

Rutting	June, 1985
3 Lift Section	
Driving Lane	.19
Passing Lane	.11
2 Lift Section	
Driving Lane	.16
Passing Lane	.06
Full Depth Section	
Driving Lane	.15
Passing Lane	.13

Bleeding

Only a small amount in the driving lane of the 3 lift section was noted in June, 1985.

Deflections

	June, 1985
3 Lift Section	.004" to .012"
2 Lift Section	.004" to .005"
Full Depth Section	.004"

There is some inconsistency in the cracking data collected to date. The section that had three lifts placed prior to the end of the paving season had 228 linear feet of cracking in April, 1984 while the 2 lift section showed 0. At this point in time there is no explanation for this. There is some indication that the June, 1985 deflections are higher in the 3 lift section, which may have contributed to the cracking. Hopefully, the performance data yet to be collected will help provide an answer. In the following year, the 3 lift section had 14 linear feet of cracking for a total of 242, while the two lift section went from 0 to 118 linear feet. To date, approximately 19% of the transverse cracks have reflected through in the test sections labeled 3 lift and 2 lift.

There was no cracking on the section north, which was all placed in one paving season. Although it has gone through only one winter, this alone appears to be enough reason to place full thickness of overlays in one paving season.

The smoothness readings are very low in the test sections and throughout the project. In some areas, such as the extension to the north, the measurement was only 0.1 inch per mile.

Rutting is a slight concern at this time. The driving lane has typically .15" to .20" rutting in the wheel tracks. The passing lane is averaging approximately .10". The Research Branch is anticipating next year's evaluation to see if this distress has stopped or is continuing to get worse.

Bleeding of the asphalt to the surface is non-existant. Only a small 3 foot strip has been recorded to date.

Finally, the deflections measured in June are quite low. They are well below the maximum allowable of .014 inches. Future measurements will be taken at the reflected transverse cracks and joints as they appear. This will provide information on the all important differential movements of the underlying slabs.

SUMMARY

1. The 5" design thickness of the overlay, placed on I-25 between SH 52 and SH 66, is a function of PCCP slab length and the temperature differential between the normal maximum in the summer and the normal minimum in the winter. Deflections are a consideration also, thicker overlays being required where deflections are higher.

2. A very important part of the design procedure is the condition survey and deflection measurements. Some care must be exercised when taking deflections. For example, time of the year and temperatures are factors which might require corrections to deflection measurements. Ideally, the best time to measure deflections on PCCP is early spring. Identification of each distress and the severity is also an important part of the condition survey.

3. Areas of severely distressed PCCP pavements were removed and replaced to help reduce deflections and minimize reflective cracking. Load transfer devices were also placed to help minimize the deflections at the transverse joints of the new and adjacent old PCCP slabs.

4. To date the performance data collected on the two test and evaluation sections established for evaluating the project indicates satisfactory performance for more than two years. Only about 19% of the transverse cracks have reflected through to the surface in the 2 and 3 lift test sections. None have appeared in the test section at the north end where paving was started and completed in one paving season and there has been no longitudinal cracking anywhere. The severity level of the cracks that have occurred is low. Deflections are currently low and bleeding or flushing of the asphalt to the

surface is non-existant. There is some rutting in the driving lane but it is too early to predict how much will eventually occur.

5. It appears that reflective cracking can be minimized by placing the full overlay thickness in one paving season as was done at the north end of the project. Allowing traffic on uncompleted overlays may accelerate reflective cracking.

6. The traffic was handled very successfully by moving it to the passing lane side and back to the driving lane as needed. As a result the cost of traffic control to the project was not very significant.

Thick structural asphalt pavement overlays on PCCP are a simple, fast, and efficient way of rehabilitating PCCP by making the old pavement smoother, safer, and stronger. The project has been successful to date. It is estimated the future testing and evaluations of the established test sections will show the long term performance to be very satisfactory.

Appendix B

I–25 Northbound from MP 235 to 240 – Rut Depths (1/100 in)																
	Feb 26, 1989				Oct 1, 1989				Mar 10, 1991				Mar 29, 1992			
	Driving Lane		Passing Lane		Driving Lane		Passing Lane		Driving Lane		Passing Lane		Driving Lane		Passing Lane	
MP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP
235.00	25	20	10	15	30	25	10	20	30	25	10	20	25	30	25	15
235.25	25	20	0	0	30	25	10	15	35	25	15	20	30	25	15	15
235.50	25	30	20	30	20	35	20	20	35	25	25	30	35	40	35	35
235.75	40	40	20	25	45	40	25	25	50	45	35	30	55	35	30	30
236.00	3	40	10	20	30	35	25	20	35	35	25	40	40	40	30	35
236.25	40	30	20	25	45	40	25	30	50	40	20	35	55	45	30	35
236.50	60	35	20	25	60	40	25	30	65	40	35	35	65	45	30	40
236.75	50	55	25	35	40	60	35	35	45	65	35	45	60	70	40	40
237.00	30	55	20	40	40	65	40	35	30	65	35	45	30	70	30	45
237.25	25	25	30	30	25	45	35	30	30	50	35	35	35	60	40	35
237.50	25	25	20	20	35	35	30	35	35	40	25	35	40	40	30	45
237.75	50	25	30	30	55	30	40	40	55	35	40	30	60	40	45	35
238.00	50	50	25	25	55	55	30	30	55	60	30	35	55	65	25	35
238.25	35	40	25	20	45	50	35	30	50	55	40	35	50	55	35	35
238.50	40	45	25	30	50	55	30	35	45	40	40	35	50	50	35	30
238.75	50	50	20	20	60	55	25	35	60	60	30	35	65	65	35	40
239.00	35	35	10	20	40	35	25	30	45	40	20	25	45	40	30	35
239.25	30	25	20	20	30	40	25	30	30	45	30	35	35	45	30	30
239.50	35	35	30	35	40	40	30	35	50	50	30	30	45	45	30	35
239.75	30	30	15	30	40	35	25	35	35	40	30	35	40	50	30	30
240.00	35	50	10	20	45	50	30	30	45	40	20	30	50	55	20	35
240.25	10	30	20	20	40	30	20	20	20	35	20	20	20	40	30	30
240.50	20	35	20	25	30	40	25	25	30	40	35	35	40	50	35	35
240.75	30	50	20	20	45	45	30	30	40	40	15	20	40	40	20	20
241.00	20	25	30	30	35	30	35	35	35	35	40	35	45	45	35	35
241.25	40	45	20	20	40	45	20	20	45	55	30	25	45	55	30	25
241.50	35	50	20	20	40	55	30	25	50	65	25	30	50	75	30	25
241.75	55	55	20	20	65	60	20	25	75	65	15	25	80	65	15	30
242.00	40	40	25	30	55	50	30	30	60	55	25	25	60	55	25	35
242.25	45	60	20	20	55	70	25	25	55	85	25	25	60	85	30	35
242.50	55	55	25	25	65	55	20	20	70	55	10	15	75	60	20	25
242.75	50	50	25	25	55	45	25	20	55	40	25	30	65	55	20	25
243.00	45	45	25	20	50	40	20	20	50	45	25	30	50	75	30	30
AVG	35.84	39.39	20.45	23.93	43.48	44.09	26.51	27.87	45.3	46.5	27.1	30.6	48.3	51.8	29.4	32.1

I–25 southbound from MP 240 to 235

Rut Depths (1/100 in)

	Feb 26, 1989				Oct 1, 1989				Mar 10, 1991				Mar 29, 1992			
	Driving Lane		Passing Lane		Driving Lane		Passing Lane		Driving Lane		Passing Lane		Driving Lane		Passing Lane	
MP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP
240.00	0	0	0	10	20	10	10	10	15	10	15	15	20	10	15	20
239.75	0	0	0	20	0	0	10	20	0	15	10	20	10	10	10	25
239.50	0	0	0	15	20	10	0	20	25	10	5	25	0	15	10	25
239.25	10	0	0	15	10	5	0	15	15	10	0	20	15	10	0	25
239.00	0	0	0	15	10	10	0	10	15	10	5	15	15	15	10	15
238.75	0	0	0	10	10	0	0	10	15	5	5	15	120	10	10	20
238.50	10	0	0	10	15	0	0	15	15	10	0	15	25	10	10	20
238.25	10	0	0	10	15	10	0	20	30	10	10	15	35	15	10	20
238.00	0	0	0	10	5	0	0	10	15	10	5	20	20	10	10	25
237.75	0	0	0	10	20	0	0	10	20	10	10	15	30	15	15	25
237.50	0	0	0	10	20	0	0	10	20	10	0	10	20	10	0	20
237.25	10	0	0	10	20	10	0	15	25	15	5	20	30	20	15	25
237.00	10	0	0	10	15	10	0	10	30	15	5	20	30	15	10	25
236.76	0	0	0	10	30	10	0	15	35	15	0	15	35	15	10	20
236.50	10	0	0	15	20	5	0	15	25	10	5	5	25	10	0	20
236.25	0	0	0	15	15	5	0	10	20	5	0	20	25	10	0	20
236.00	10	0	0	15	20	5	0	15	30	15	0	20	35	15	0	20
235.75	10	0	0	15	25	10	0	20	30	15	0	15	30	10	0	25
235.50	10	0	0	20	15	5	0	20	25	10	0	25	25	10	10	25
235.25	0	0	0	10	10	5	0	10	10	5	0	15	10	0	0	10
235.00	10	0	0	20	15	10	0	15	20	15	10	15				
AVG	4.761	0	0	13.09	15.71	5.714	0.952	14.04	20.7	11.0	4.3	16.9	27.8	11.8	7.3	21.5

I-25 Average Rutting (1/100 in)				
Northbound from MP 235 to 243				
	Driving Lane		Passing Lane	
DATE	RWP	LWP	RWP	LWP
May-84	0	0	0	0
Jun-85	19	19	11	11
Feb-89	36	39	20	24
Oct-89	43	44	27	28
Mar-91	45	47	27	31
Mar-92	48	52	29	32

FIG 6 DATA

I-25 Average Rutting (1/100 in)				
Southbound from MP 240 to 235				
	Driving Lane		Passing Lane	
DATE	RWP	LWP	RWP	LWP
Oct-88	0	0	0	0
Feb-89	5	0	0	13
Oct-89	16	6	1	14
Mar-91	21	11	4	17
Mar-92	28	12	7	22

FIG 7 DATA

I-25 Average Rutting (1/100 in)				
Test section averagess				
I25 S B from MP 236 to 237				
	Driving Lane		Passing Lane	
DATE	RWP	LWP	RWP	LWP
Oct-88	0	0	0	0
Feb-89	10	0	0	8
Oct-89	24	10	1	15
Mar-91	30	14	5	18
Mar-92	34	17	6	22

FIG 8 DATA

I-25 Average Rutting (1/100 in)				
Control section averages				
I25 Southbound 236 to 237				
	Driving Lane		Passing Lane	
DATE	RWP	LWP	RWP	LWP
Oct-88	0	0	0	0
Feb-89	14	1	0	9
Oct-89	22	6	0	14
Mar-91	26	9	2	17
Mar-92	27	12	6	22

FIG 9 DATA

I 25 Southbound Test Section – Rut Depths (1/100 in)																
	Feb 26, 1989				Oct 1, 1989				Mar 10, 1991				Mar 29, 1992			
	Driving Lane		Passing Lane		Driving Lane		Passing Lane		Driving Lane		Passing Lane		Driving Lane		Passing Lane	
MP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP
236.80	20	0	0	10	35	15	0	15	40	10	5	20	45	20	10	25
236.78	0	0	0	10	15	5	0	15	20	10	5	15	25	10	10	20
236.76	0	0	0	10	30	10	0	15	35	15	0	15	35	15	10	20
236.74	0	0	0	10	15	5	0	15	25	10	5	15	25	15	0	20
236.72	10	0	0	10	20	10	0	15	30	20	10	20	35	20	10	20
236.70	10	0	0	10	30	15	10	10	35	10	5	15	40	15	0	15
236.68	20	0	0	10	25	10	0	10	35	15	0	15	40	25	10	25
236.66	20	0	0	0	30	10	0	15	30	20	5	20	35	20	0	25
236.64	10	0	0	10	20	10	0	20	30	15	5	20	30	10	10	20
236.62	10	0	0	0	20	10	0	15	20	15	5	20	30	20	0	25
AVG	10	0	0	8	24	10	1	14.5	30.0	14.0	4.5	17.5	34.0	17.0	6.0	21.5

I 25 Southbound Control Section – Rut Depths (1/100 in)																
	Feb 26, 1989				Oct 1, 1989				Mar 10, 1991				Mar 29, 1992			
	Driving Lane		Passing Lane		Driving Lane		Passing Lane		Driving Lane		Passing Lane		Driving Lane		Passing Lane	
MP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP	RWP	LWP
236.5	10	0	0	15	20	5	0	15	25	10	5	5	25	10	0	20
1+00	10	0	0	10	25	5	0	10	30	10	5	20	25	10	10	25
2+00	10	0	0	0	25	5	0	15	25	10	0	15	30	15	0	20
3+00	15	10	0	10	20	10	0	15	25	10	5	20	25	10	10	25
4+00	15	0	0	0	15	5	0	10	20	10	0	15	25	15	10	20
5+00	20	0	0	15	25	10	0	20	25	10	0	20	30	15	10	25
6+00	15	0	0	15	25	5	0	15	30	5	0	25	30	10	0	20
AVG	13.6	1.4	0.0	9.3	22.1	6.4	0.0	14.3	25.7	9.3	2.1	17.1	27.1	12.1	5.7	22.1