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Evaluation of the Performance, Cost-Effectiveness, and Timing of Various Pavement Preservation Treatments

Scott Shuler, Ph. D., P. E.

October 2010

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16. Abstract

This research study evaluated the performance of various pavement preservation treatments over time and under different environmental conditions to quantify the economics of each treatment type. There are three primary techniques utilized in Colorado for preservation of asphalt pavements and three for concrete pavements. For asphalt pavements these are crack sealing, chip seals, and thin hot mix asphalt overlays. For concrete pavements the treatments are joint resealing, cross-stitching, and microgrinding. Full-scale test sections were constructed in 2005 and some additional test sections previously constructed were also included for performance measurement. These test sections include crack sealing, chip seals, thin overlays, joint resealing, cross-stitching and microgrinding.

Test results indicate that continuation of crack sealing, chip sealing, and thin hot mix overlays is justified for asphalt pavements. Performance results of the preservation treatments for concrete pavements were not as successful. Propagation of the crack into the adjacent slab occurred in the cross-stitching test sections. Microgrinding concrete pavements does not appear to be effective at reducing cracking and may be detrimental to performance. The disappointing results of the joint resealing test sections indicate that a review of the specifications should be considered.

Implementation:
The *Preventive Maintenance Best Practices Manual* is Appendix A of this report. The methods described in this manual are based on a review of the literature, a series of interviews conducted in each Colorado DOT region, full-scale field test sections, and experience of the researchers. These best practices should be followed when preventive maintenance procedures are conducted on asphalt and concrete pavements in Colorado.

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Colorado State University

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EXECUTIVE SUMMARY

This research evaluated the performance of various pavement preservation treatments over time and under different environmental conditions to evaluate the economics of each treatment type. There are three primary techniques utilized in Colorado for preservation of asphalt pavements and three for concrete pavements. For asphalt pavements these are crack sealing, chip seals, and thin hot mix asphalt (HMA) overlays. For concrete pavements the treatments are joint resealing, cross-stitching, and microgrinding. Full-scale test sections were constructed in 2005 and 2006 and some additional thin stone matrix asphalt (SMA) and ultra-thin bonded wearing course test sections constructed previous to this study in 2004 were also included for measurement of performance.

Variables evaluated for crack sealing included two sealants, two climates, and presence or absence of a deicing chemical prior to sealing operations. Chip seal variables include chip size, gradation and climate. Thin overlays include dense graded hot mix asphalt, ultra-thin bonded wearing course, and stone matrix asphalt applied over both asphalt and concrete pavements. Cross-stitching of concrete was done using two methods including conventional deformed reinforcing steel and fiberglass panels.

Results of the crack seal experiment indicate a significant difference in performance between the two products tested and that magnesium chloride applied to the pavement prior to sealing operations may actually improve performance at higher elevations. The average time for longitudinal and transverse cracking to return to pretreatment levels for the thin SMA test sections was 2.1 and 3.3 years, respectively. For chip seals transverse cracking returns to pre-treatment levels in less than 1 to 3.5 years and from less than 1 to 6 years for longitudinal cracking. One test pavement evaluated the effects of fog sealing the newly constructed chip seal at low and high elevations. Results indicate the fog sealed sections at the low elevation perform better than the non-fog sealed sections with respect to cracking, but at the higher elevation the non-fog sealed sections performed better with respect to longitudinal cracking. Thin, 1-inch thick, hot mix asphalt overlays provided approximately 1.5 years of good performance before returning to pre-treatment cracking levels. However, one location where hot in-place recycling (HIPR) under 1.5 and 2 inch overlays was evaluated, is just beginning to show signs of cracking

in the control sections after 3 years where HIPR was not utilized. The test sections where overlays were placed over HIPR has not cracked after 3 years. Test sections where microgrinding of concrete pavement was compared with control sections without grinding resulted in poorer performance in the microgrind sections with respect to all forms of cracking. Results of the cross-stitching experiment indicates that both deformed bars and fiberglass panels perform equally well at maintaining crack width but the repaired crack has propagated into the adjacent concrete slab. The sealant used in the joint resealing experiment became brittle and began to fail after three years.

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1.0 INTRODUCTION

This research is intended to determine the most economical means of preserving asphalt and concrete pavements in Colorado. The process used to accomplish this included a survey of current published literature and interviews with individuals responsible for pavement preservation, installation of experimental test pavements to measure performance under local conditions and recommendations based on the findings. This report documents the results of a five year study which included a literature review, interviews with maintenance and construction personnel in all six regions, installation of pavement test sections and monitoring of the performance of these pavements.

2.0 PAVEMENT PRESERVATION PRACTICES IN COLORADO

Interviews were conducted with CDOT maintenance and construction personnel to determine the current methodology used for implementing pavement preservation. A summary of the current methods used, specific methodology for the methods and the decision process for implementing pavement preservation is shown in Table 1.

Table 1. Interview Results: Pavement Preservation Practices in Colorado in 2005

Current Methods Used	**Specific Methods**	**Decision Process**
1. Crack Seal (No Routing) 2. Chip Seal 3. Thin Overlay (<1.5in) includes any hot mix application 4. Slab Replacement 5. Joint Resealing 6. Microgrinding	Crack Seal 1. < ¼ inch wide cracks 2. Early winter/spring 3. Blow out cracks with compressed air 4. Heat lance used by some regions, however, one region indicated it was not effective if $MgCl2_2$ was present. 5. CRS and blotter sand used by one region. 6. Hot pour sealants used by most regions a. Deery-D3405- will not stay in cracks if $MgCl2_2$ has been used. However, one region indicated that if crack pouring is done in fall season before $MgCl2_2$ use, the Deery product works acceptably. b. Asphalt rubber, (Meggison) – stays in cracks better if $MgCl2_2$ has been used 7. Bump occurs in thin overlays if O/L placed before 1 year 8. Asphalt rubber causes hump in crack as pavement temperature rises 9. Start on Nov 15, on or between snow storms Chip Seal 1. 3/8 in; HFMS-2P 2. Asphalt vendor takes chips and determines 'compatibility' with chips and supplies spread rate. 3. MS used because of increased time allowed before set 4. < 3000 ADT because of windshield damage 5. Estimated life is approx 5 yrs 6. 1/year for 'worse first' 7. HFMS-more forgiving than RS 8. $1/sy (DOT); $1.60 (Contract) 9. Double chip seals in one region Fog Seals 1. Use varies with Region Thin O/L 1. 1 inch or less because of cost 2. Improve rideability/rut filling 3. Use whatever plant can supply that meets specs. SX with AC5 or AC10. No PG grades, yet. 4. One roller-10-15 ton vibratory/non-vibratory 5. Novachip - Best, but cannot afford if project is too small PCC 1. R&R backer rod and reseal joints	1. Road inventory randomly selected in one-mile increments. Budget identified based on issues in these segments. 2. TM2[1] identifies distress and reports to TM3 or LTC. LTC decides what to repair and how in early spring and fall seasons. 3. Preventive Maintenance-Do Something before 3 years old. 'Something' is usually: crack seal as cracks appear then chip seal if traffic is appropriate $150k spending limit if maintenance doing work. Coordinate with Engineering so Maintenance treatments do not get overlaid. 4. Four triggers from pavement management program: Age Rutting Cracking ADT 5. Crack seal when cracks get to about ¼ in - 9 to 12 mos before overlay to prevent bleed-through 6. Chip seal with DOT traveling crew 1 or 2 summers after crack sealing 7. Apply chip seals from 2.5-3 yrs after overlay applied 8. Expect 5 yrs service from chip seals 9. $150k limit is difficult to do enough work in some areas 10. With 5 yr window, this keeps them from sealing everything that is needed 11. Use pavement management (PM) program with judgment 12. Some believe PM program has flaws in triggers used to identify repairs 13. Better coordination needed with engineering to keep maintenance projects from getting overlaid next year. 14. Fix worse first (about 50-75% of budget) 15. Pavement preservation(25-50% of budget)
Methods Eliminated		
1. Reclamite 2. Sand Seal – Believe contributes to rutting 3. Cold Patching 4. Fog Seals – Use varies		

* Note: Columns in Table 1 are independent of each other and do not necessarily relate directly.

[1] See Figure 1 for organizational chart.

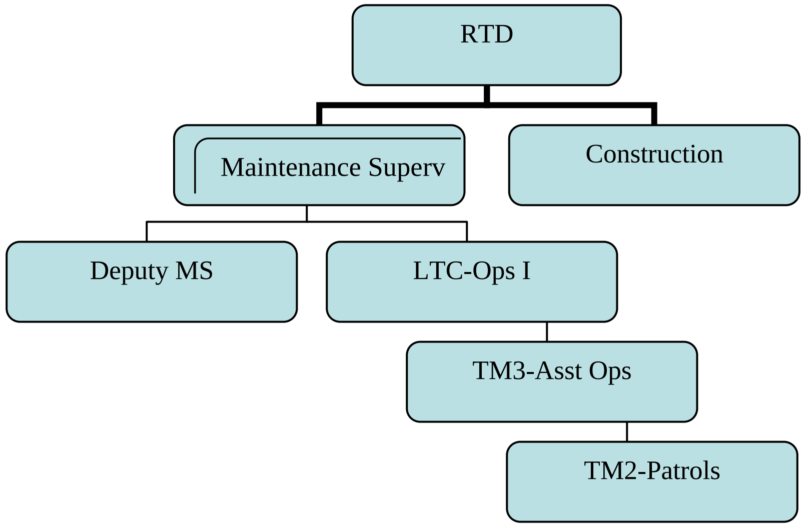


Figure 1. Organizational Structure for CDOT Region Maintenance

2.1 Literature Review

There is a significant volume of information available regarding pavement preservation. The notion of applying incremental treatments to a pavement to extend serviceability is key to the concept of preventive maintenance. These incremental treatments are optimized when applied at the correct time. The correct time varies with traffic loading, pavement age, weather, materials, design, and construction quality (Peterson, 1981). However, most agree that pavement preservation should be applied during the period when the pavement remains in good condition as shown in Figure 2 (Peterson, 1981) estimating that funds spent early in the life of the pavement will return significant cost savings (Johnson, 1983) as shown in Figure 3.

Routine maintenance and pavement preservation are often confused and several definitions can be found in the literature. For example, routine maintenance is often synonymous with 'reactive' maintenance. 'Reactive' maintenance activities for pavements include pothole

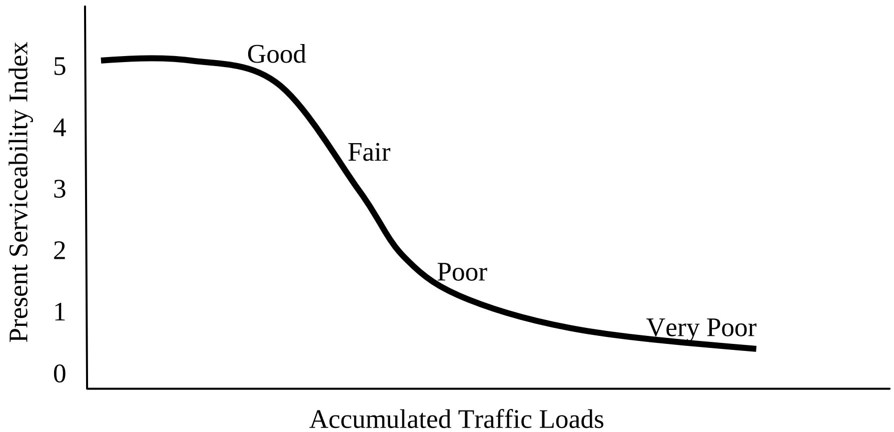


Figure 2. Typical Pavement Performance Curve, after Carey (1960)

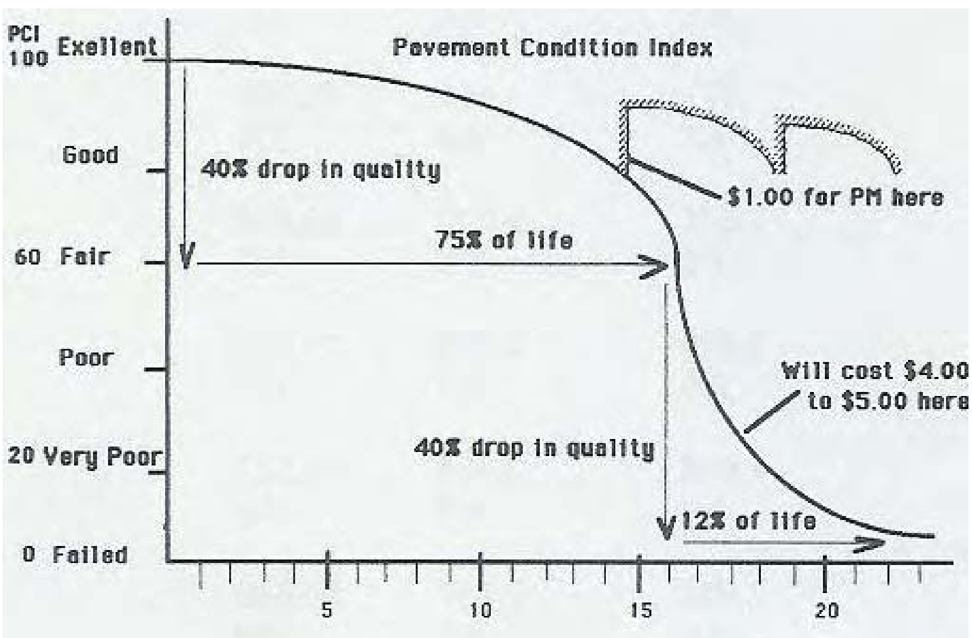


Figure 3. Typical Pavement Life Cycle (from O' Brien, L. G., 1989)

repair, blowup repair, and spall repair. Pavement preservation is a programmed strategy intended to arrest light deterioration, retard progressive failures and reduce the need for 'reactive' maintenance. Pavement preservation is usually a cyclic, planned event. Pavement preservation is generally considered to not significantly improve load-carrying capacity but instead to extend the useful life of the pavement. Typical activities include crack and joint sealing, joint repair, limited slab replacement, undersealing or mudjacking, surface treatments, grinding, machine-laid patching (O' Brien, L. G., 1989).

A survey of pavement preservation practices (Zimmerman, 1995) indicated that strategies differ depending on the needs and objectives of each agency and that not one method is best suited to all agencies, as might be expected. This study found that the simplest methods of pavement management were practiced by most highway agencies surveyed. The study found that pavement condition surveys rather than more complex priority assessment models or network optimization models were being used.

The Strategic Highway Research Program SPS-3 and SPS-4 research studies included both asphalt and concrete pavement preservation treatments including slurry seal, chip seal, thin overlays, crack sealing and joint resealing. Expert Task Groups (ETG) evaluated the performance of the test sections and a report (Morian, 1997) summarizing the ETG findings concluded that:

- Pavement preservation treatments generally outperformed control sections
- Treatments applied to pavements in good condition have shown good results
- Traffic level and pavement structural adequacy did not appear to affect performance

A detailed analysis of the Long-term Pavement Performance (LTPP) data from the SPS sites through 2001 was accomplished by NCHRP Project 20-50(03/04) and reported by Hall, et al (2004). This report indicates that with respect to roughness, rutting and fatigue cracking, thin overlays were most effective followed by chip seals then slurry seals. As pavement roughness increased there was some evidence that chip seals had an effect on long-term roughness. Slurry seals showed no effect on long-term roughness. The thin overlays, as expected, were the only treatment to affect long-term rutting. Fatigue cracking is significantly less in the thin overlay sections than corresponding control sections. The chip seal and slurry seal sections had less cracking than the control sections, as well. However, long-term cracking has accelerated for both chip seal and slurry seal sections. Crack sealing did not reduce long-term cracking with more cracking occurring in test sections with crack sealer than control sections without crack sealer. However, the report indicates that this may not be due to adverse effects of crack sealing, but rather to 1) sealing of new cracks (cracks that appeared after the initial treatment date), and/or 2) the greater visibility of sealed cracks. The study concluded that with respect to IRI reading, rutting and cracking on concrete pavements in the SPS-6 study that except for an 8 inch and 4 inch asphalt overlay on a cracked and seated concrete pavement that diamond grinding, full-depth repair and joint and crack sealing had the next best effect. It is interesting that no added benefit was associated with subdrainage improvement, load transfer restoration or undersealing.

Research (Peshkin, et al, 2004) indicates pavement preservation treatment timing was based on: 1) predetermined schedules, 2) time since a previous maintenance and rehabilitation event, 3) maintenance surveys, and 4) pavement management systems. One result of this research is analytical software called OPTime based on Microsoft® Excel which provides a means for identifying the optimal time to apply various pavement preservation treatments.

There are many guidelines available to determine when to apply pavement preservation treatments. Colorado maintenance personnel sometimes refer to the 'Shaffer Memo' (Shaffer, 1991) developed by Doug Shaffer of CDOT to provide guidance to maintenance forces regarding crack filling and joint resealing operations. A more recent publication developed for CDOT (CDOT, 2004) provides guidelines for crack sealing, crack filling, sand seals, chip seals, micro-surfacing, thin bonded wearing courses, thin overlays (less than 1-1/2"), surface milling with non-structural HMA overlay (less than 1-1/2"), diamond grinding, concrete crack sealing, concrete joint resealing, partial depth concrete pavement repair, dowel bar retrofit, and full depth concrete pavement repair. This document also provides guidance regarding the expected life extension provided by each treatment when applied under appropriate conditions.

3.0 TEST SECTION CONSTRUCTION

Full-scale pavement test sections were installed and the condition monitored over time so the performance of the preservation treatments could be quantified. Treatments placed on asphalt pavements include crack sealing, chip seals, thin hot mix asphalt overlays and thin stone matrix asphalt (SMA). Treatments applied to concrete pavements include joint resealing, cross-stitching and microgrinding. Treatments placed prior to this research which were also monitored include SMA and ultra-thin bonded wearing courses (UTBWC) applied to asphalt pavements.

Tables 2 and 3 summarize each treatment studied for preservation of asphalt and concrete pavements in this research, respectively.

Table 2. Preservation Treatments Evaluated on Asphalt Pavements

Pavement	Treatment	Location	Installation
Asphalt	Crack Seal	SH66, Lyons	May 4, 2005
		SH7, Estes Park	May 5, 2005
	SMA	SH58, Evergreen	July 1, 2004
		SH13, Rifle	June 14, 2005
		I70, Glenwood Canyon	July 6, 2005
	Chip Seal	US34, Drake	July 18, 2005
		SH14, Briggsdale	July 14, 2005
		US285, Poncha Pass/Springs	June 5, 2006
		US24, Leadville	July 11, 2006
		US50, Swink	July 24, 2006
	Thin HMA	US6, Golden	June 23, 2005
		SH121, Littleton	May 16, 2006
		US550, Durango	May 25, 2006
		US550, Coal Bank Pass	May 30, 2006
		US40, Golden	May 3, 2006
	UTBWC	SH58, Golden	June 15, 2005
		Table Mesa Rd, Boulder	July 8, 2004

Table 3. Preservation Treatments Evaluated on Concrete Pavements

Pavement	Treatment	Location	Installation
Concrete	Microgrinding	I70, Rifle	Sept 13, 2005
	Cross-stitching	US287, Campo	May 12, 2005
	Joint Resealing	US287, Campo	May 12, 2005

3.1 Crack Seal

Two sites were established in 2005 to evaluate crack seal performance. These sites are located on SH 7 south of Estes Park and SH66 east of Lyons. The SH66 site is located at an elevation of 1635 meters (5364 feet). SH7 is located at an elevation of 2294 meters (7526 feet).

The purpose of this part of the study was to assess the effect of magnesium chloride ($MgCl_2$) application on the performance of two types of crack sealants at two elevations in Colorado. The literature review did not identify any studies evaluating the association between pavement preservation crack fill materials and exposure to $MgCl2_2$. Concerns regarding sealant integrity when $MgCl_2$ residue is present in cracks was expressed by maintenance personnel during the interview segment of this project. Therefore, this portion of the research project was designed to evaluate potential effects of $MgCl_2$ on crack seal performance.

The experiment was designed to evaluate the association between crack seal remaining over a three year period and two different crack sealants, with and without exposure to $MgCl_2$.

A digital distance measuring wheel was used to measure the length of crack seal remaining in each crack over a three year period. Measurements of remaining crack seal were made two times per year from spring 2005 to spring 2008. Measurement of each crack begins from the white line painted on the roadway near the shoulder of the pavement and continues to the yellow line marking the center of the roadway. Measurements were performed using the same technique each time.

3.1.1 SH 66, Lyons

These test sections were placed by CDOT maintenance personnel on May 4, 2005. They are located on SH 66 in the westbound lane beginning at Milepost 31 (Station 0+00). Two types of crack filler were applied. These are from Deery, Inc. and Meggison Enterprises, Inc. The pavement was sprayed with $MgCl_2$ deicing solution prior to crack filling for half of the sections and no $MgCl_2$ was applied in the other sections. The resulting test pavement contains 2 crack fillers x 2 deicing applications plus 2 controls x 2 replicates = 10 test sections as shown in Table 4. There are six cracks treated in each section. This results in 60 cracks for performance evaluation over a distance of 891 feet from Milepost 31.

Table 4. Crack Fill Test Sections on SH 66, Lyons

Section No.	Crack No.	Dist (Ft)	Crack Fill Type	$MgCl_2$ Applied?
10	-	0.00		None
	1	13.42	Deery	
	2	43.42		
	3	63.83		
	4	85.50		
	5	127.33		
	6	137.50		
9	7	154.00	Megg	
	8	161.33		
	9	171.50		
	10	181.25		
	11	195.17		
	12	207.00		
8	13	225.92	Deery	
	14	234.83		
	15	247.00		
	16	253.25		
	17	262.67		
	18	277.25		
7	19	290.25	Megg	
	20	297.33		
	21	308.58		
	22	320.58		
	23	340.50		
	24	349.67		
6	25	365.50	None	
	26	370.25		
	27	379.83		
	28	396.42		
	29	404.58		
	30	455.50		
5	31	467.43	Deery	Yes
	32	478.92		
	33	491.08		
	34	500.58		
	35	514.92		
	36	541.00		
4	37	561.00	Megg	
	38	574.67		
	39	587.08		
	40	602.00		
	41	613.83		
	42	623.17		
3	43	633.50	Deery	
	44	653.17		
	45	616.83		
	46	689.75		
	47	701.75		
	48	716.00		
2	49	730.92	Megg	
	50	745.43		
	51	758.00		
	52	771.33		
	53	784.92		
	54	825.08		
1	55	831.58	None	
	56	839.50		
	57	858.08		
	58	871.17		
	59	882.75		
	60	891.00		

3.1.2 SH 7, Estes Park

These test sections were placed by CDOT maintenance personnel on May 5, 2005. They are located on SH7 in the southbound lane beginning at Milepost 3 (Station 0+00). Two types of crack filler were applied from suppliers Deery and Meggison. The pavement was sprayed with $MgCl_2$ deicing solution prior to crack filling for half of the sections and no $MgCl_2$ was applied in the other sections. The resulting test pavement factorial experiment has 2 crack fillers x 2 deicing applications x 2 replicates = 8 test sections as shown in Table 5. There are six cracks treated in each section with six adjacent control

Table 5. Crack Fill Test Sections on SH7, Estes Park

Section No.	Crack No.	Dist (Ft)	Crack Fill Type	$MgCl_2$ Applied?
4	49	946.83	Deery	Yes
	50	1002.08	None	
	51	1015.00	Deery	
	52	1033.50	None	
	53	1048.42	Deery	
	54	1078.00	None	
	55	1177.33	Deery	
	56	1244.58	None	
	57	1265.33	Deery	
	58	1285.08	None	
	59	1297.83	Deery	
	60	1309.25	None	
3	61	1318.08	Megg	
	62	1326.50	None	
	63	1337.42	Megg	
	64	1343.17	None	
	65	1348.50	Megg	
	66	1364.00	None	
	67	1371.08	Megg	
	68	1419.00	None	
	69	1429.00	Megg	
	70	1451.67	None	
	71	1460.75	Megg	
	72	1476.75	None	
2	73	1491.25	Deery	
	74	1506.83	None	
	75	1515.92	Deery	
	76	1547.00	None	
	77	1574.58	Deery	
	78	1581.67	None	
	79	1593.33	Deery	
	80	1609.00	None	
	81	1627.67	Deery	
	82	1653.92	None	
	83	1677.33	Deery	
	84	1685.00	None	
1	85	1703.00	Megg	
	86	1718.83	None	
	87	1729.08	Megg	
	88	1738.58	None	
	89	1753.17	Megg	
	90	1760.58	None	
	91	1776.00	Megg	
	92	1792.00	None	
	93	1802.00	Megg	
	94	1817.75	None	
	95	1835.67	Megg	
	96	1858.75	None	

Section No.	Crack No.	Dist (Ft)	Crack Fill Type	$MgCl_2$ Applied?
8	-	0.00		No
	1	15.25	Deery	
	2	28.25	None	
	3	37.25	Deery	
	4	52.67	None	
	5	64.17	Deery	
	6	80.08	None	
	7	90.33	Deery	
	8	106.67	None	
	9	121.83	Deery	
	10	135.08	None	
	11	153.67	Deery	
	12	182.08	None	
7	13	197.25	Megg	
	14	205.75	None	
	15	230.92	Megg	
	16	261.00	None	
	17	290.00	Megg	
	18	318.58	None	
	19	368.42	Megg	
	20	391.58	None	
	21	412.67	Megg	
	22	426.92	None	
	23	441.67	Megg	
	24	451.00	None	
6	25	468.33	Deery	
	26	496.58	None	
	27	514.00	Deery	
	28	540.75	None	
	29	553.92	Deery	
	30	584.42	None	
	31	610.92	Deery	
	32	631.33	None	
	33	651.42	Deery	
	34	672.08	None	
	35	683.33	Deery	
	36	708.50	None	
5	37	724.33	Megg	
	38	740.00	None	
	39	749.92	Megg	
	40	772.17	None	
	41	796.75	Megg	
	42	830.75	None	
	43	860.00	Megg	
	44	872.17	None	
	45	884.50	Megg	
	46	891.25	None	
	47	917.42	Megg	
	48	939.00	None	

cracks where no crack filler was applied. This results in twelve cracks per section or 96 cracks for performance evaluation over a distance of 1858.75 feet from Milepost 3.

3.2 SMA

3.2.1 SH 74, Evergreen

A stone matrix asphalt surface was placed by Asphalt Paving Company in June, 2004 on SH 74 north of Evergreen. Two test sections were established on July 7, 2005 in the southbound driving lanes as shown in Figure 4.

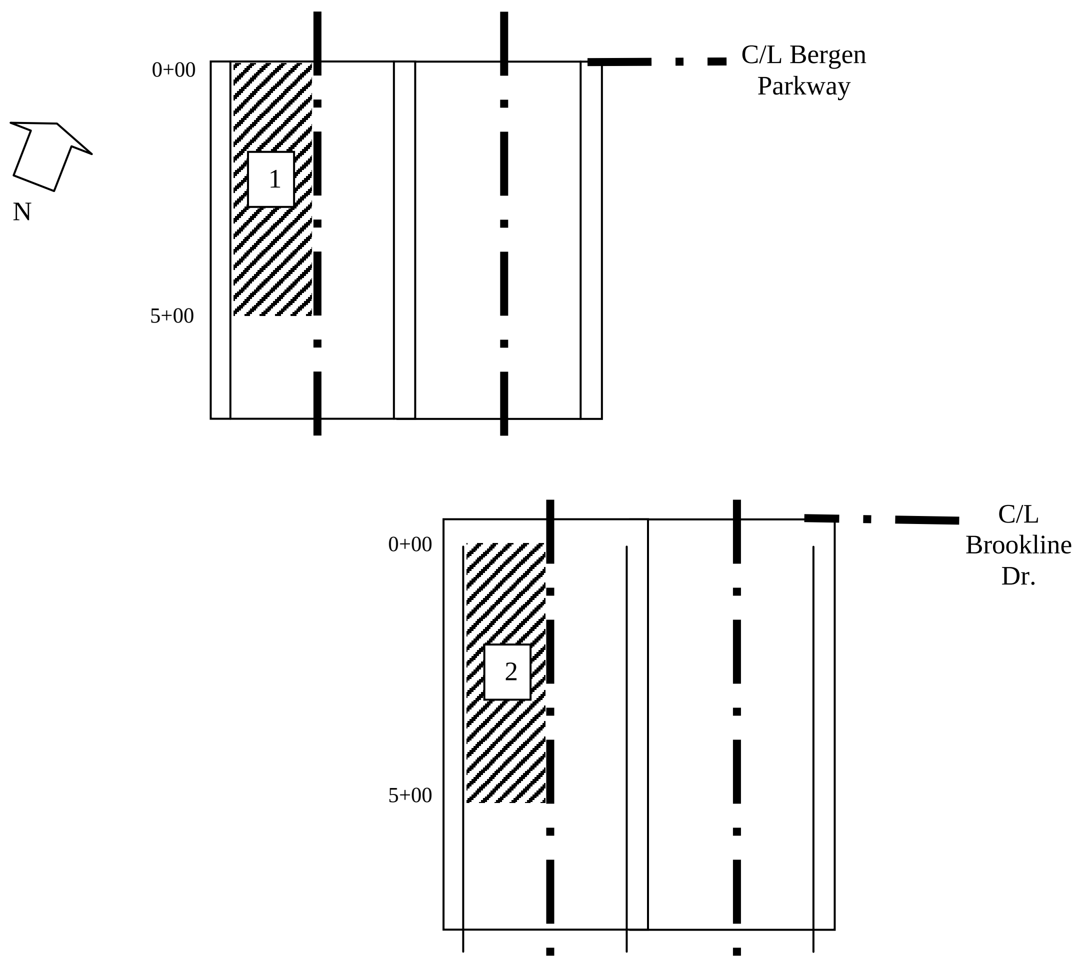


Figure 4. SMA Test Sections on SH 74, Evergreen

3.2.2 SH 13, Rifle

A stone matrix asphalt overlay was placed by United Companies on June 14, 2005 on the SH 13 by-pass west of Rifle. One week prior to the overlay four test sections were established in the locations shown in Figure 5.

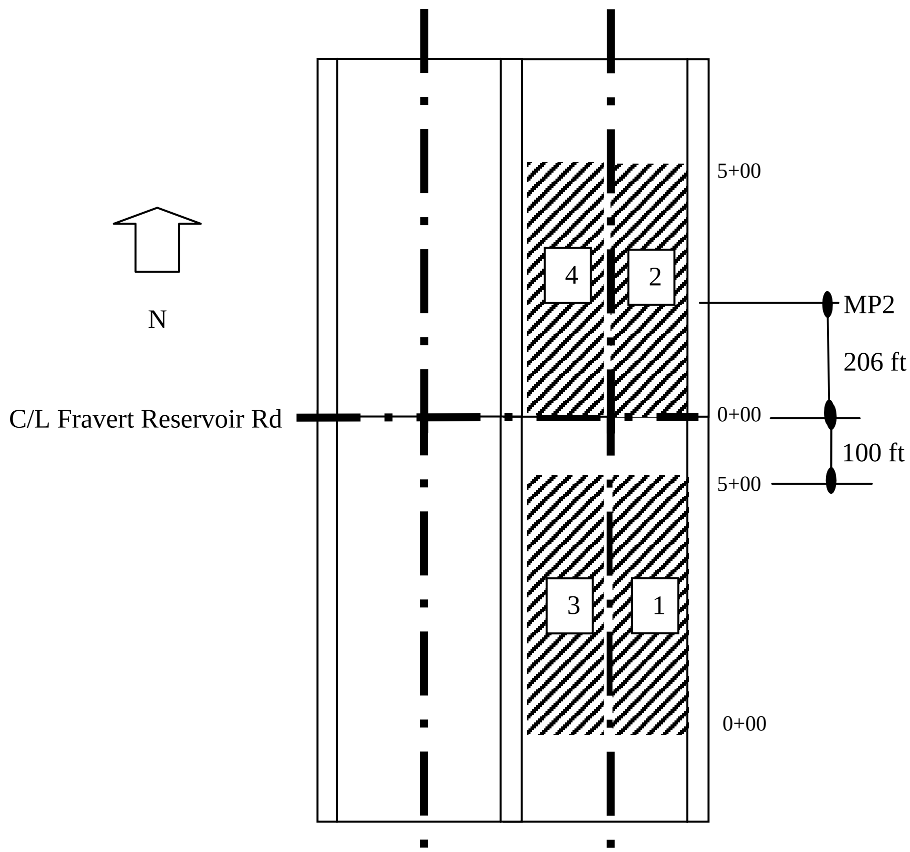


Figure 5. SMA Test Sections on SH13, Rifle

3.2.3 I-70, Glenwood Canyon

A stone matrix asphalt surface was placed by United Companies on July 6, 2005 on the east and westbound lanes of I-70 at approximately Milepost 125. Two evaluation sections were established in the eastbound lanes and two sections were established in westbound lanes as shown in Figure 6 prior to construction.

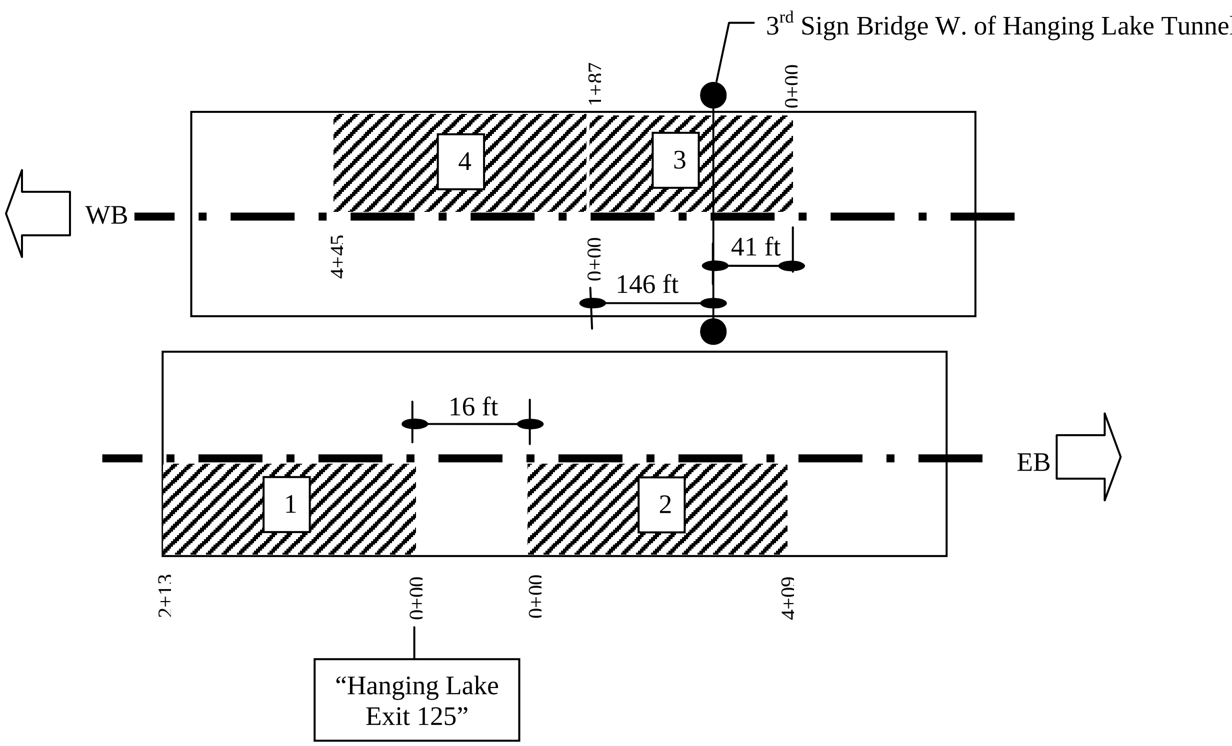


Figure 6. SMA Test Sections on I70, Glenwood Canyon

3.3 Chip Seals

Five sites were established to evaluate chip seal performance. Two of these sites were constructed in 2005 and three were constructed in 2006.

3.3.1 US 34, Drake

This chip seal was placed on US 34 east of Drake on July 18, 2005 by A-1 Chip Seal. Four 500 foot test sections were included as part of this research. The test sections are located in the eastbound and westbound lanes as shown in Figure 7. Chips used conform to the average gradation shown in Table 6.

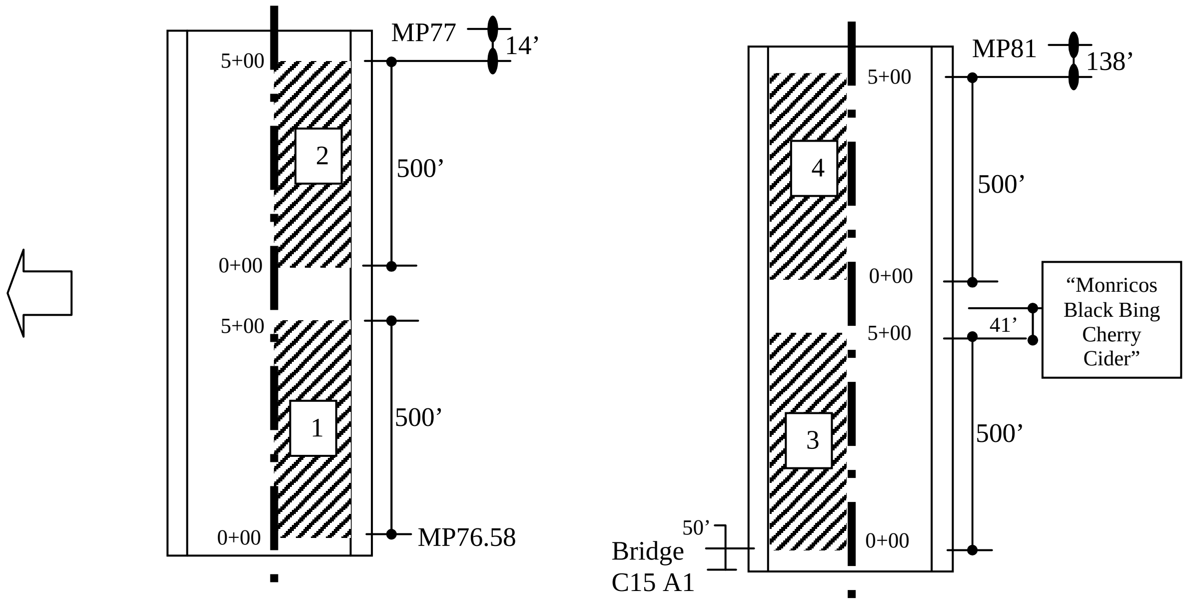


Figure 7. Chip Seal Test Sections US34 Drake

Table 6. Gradation of Aggregate Chips on US34, Drake

Sieve	Passing, %
3/8”	100
No. 4	5
No. 200	0.9

3.3.2 SH 14, Briggsdale

A chip seal was placed on SH 14 east of Briggsdale on August 22, 2005 by A-1 Chip Seal. Four 500-foot test sections were included as part of this research. The test sections are located in the eastbound and westbound lanes as shown in Figure 8. Chips used in the test sections are shown in Table 7. Emulsion properties are shown in Table 8. Materials application rates are shown in Table 9.

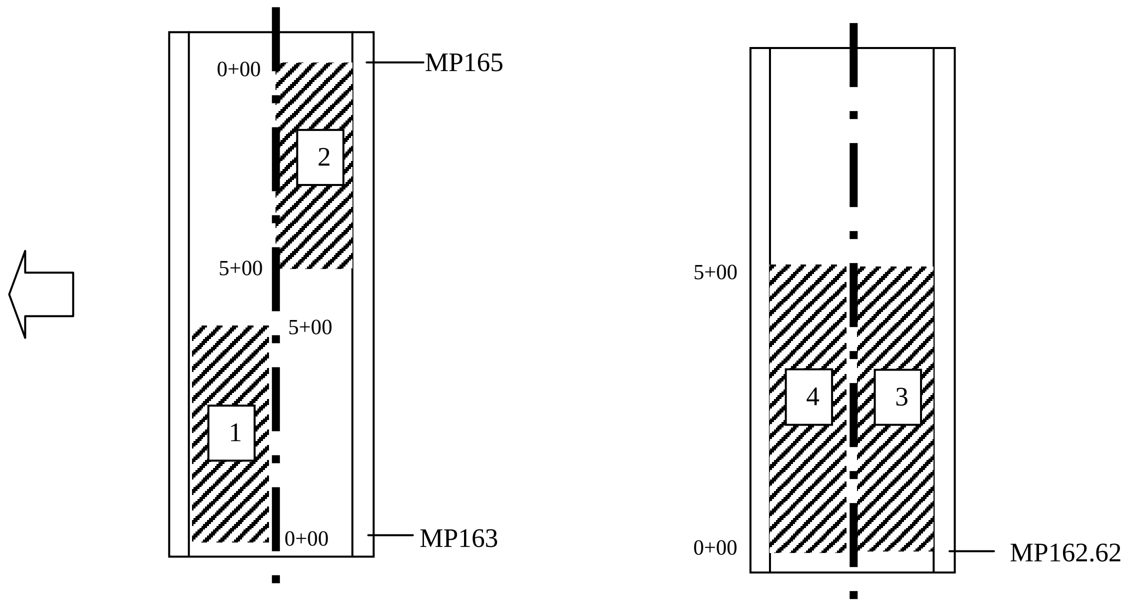


Figure 8. Chip Seal Test Sections SH14 West of Briggsdale

Table 7. Gradation of Aggregate Chips on SH14, Briggsdale

Sieve	Passing, %	
	Sections 1 and 2	Sections 3 and 4
½”	100	
3/8”		100
5/16”	14	
No. 4	2	5
No. 8	1	
No. 30	1	
No. 50	1	
No. 100	1	
No. 200	0.9	0.9

Table 8. Emulsion Properties on SH 14, Briggsdale

Viscosity SSF, @ 122 F., sec.	AASHTO T 59	150
Sieve Test, %	AASHTO T 59	0.1
Particle Charge	AASHTO T 59	Positive
Oil distillate by volume of emulsion, %	AASHTO T 59	1
Residue by Evaporation, %	Appendix D	65
Penetration, 77F,, 100g, 5s	AASHTO T49	120
Ductility, 77F, 5cm/min, cm	AASHTO T5	100
Torsional Recovery, %	CT-332	20
Toughness, in-lbs	CPL-2210	70
Tenacity, in-lbs	CPL-2210	45
Elastic Recovery, %	CPL-2211	60

Table 9. Materials Application Rates for SH 14, Briggsdale

Test Section	Chip	Chip Rate, lbs/sq-yd	CRS-2P, gal/sq-yd
1	½”	35	0.48
2	½”	35	0.48
3	3/8” ‘Drake’	28	0.42
4	3/8” ‘Drake’	28	0.42

3.3.3 US 285, Poncha Springs and Pass

A chip seal was placed on US 285 on June 5, 2006 by United Companies from the Arkansas River Bridge to the top of Poncha Pass. Ten 500-foot test sections were established near the Arkansas River Bridge and ten were established at the top of Poncha Pass as shown in Figure 9. The test sections were installed to measure the effects of using a fog seal over the chip seal and to measure the effect of removing the thermoplastic striping prior to chip sealing in an attempt to alleviate chip loss due to delamination of the thermoplastic after chip sealing.

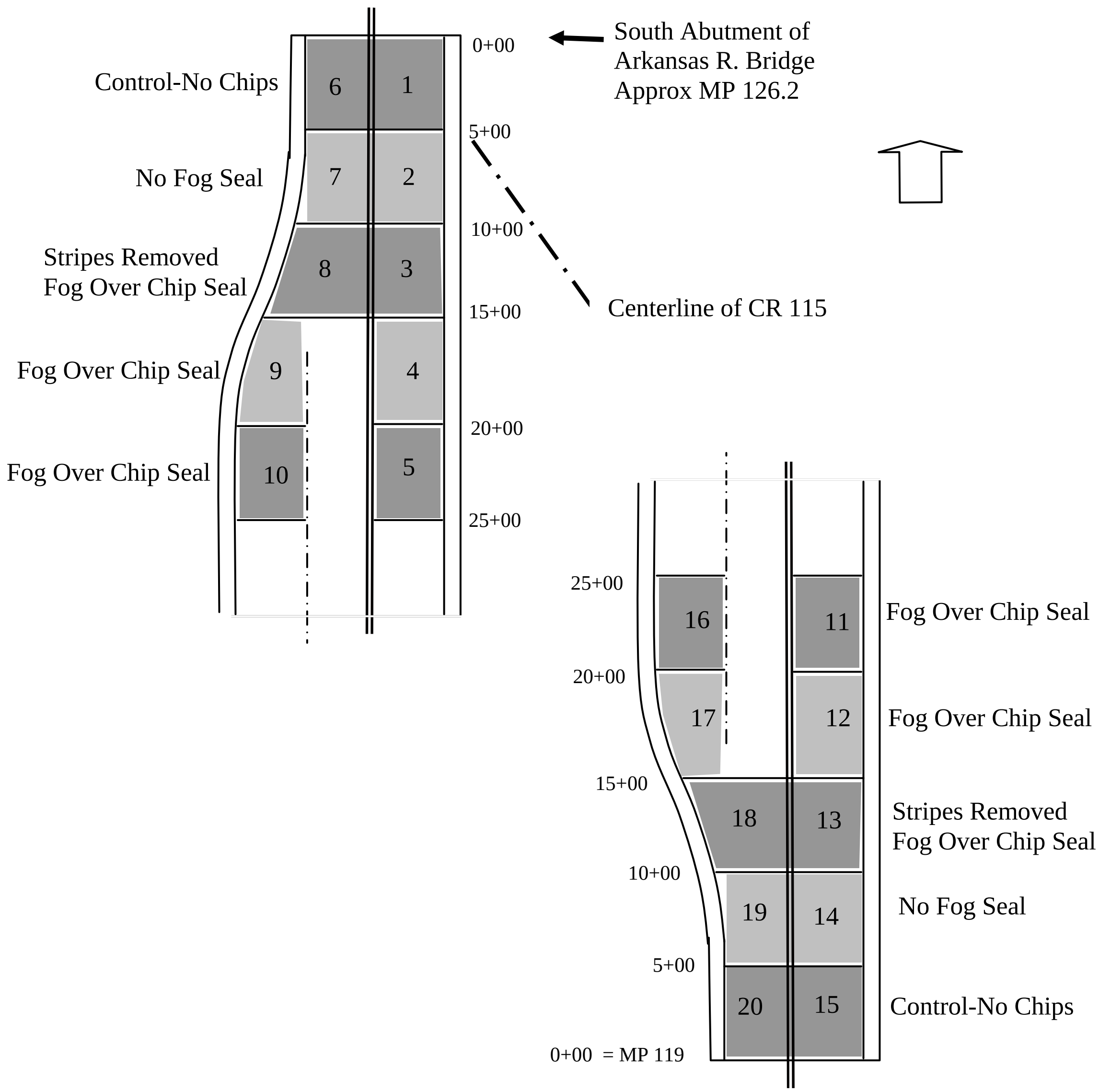


Figure 9. Chip Seal Test Sections on US 285 Poncha, Springs/Pass

3.3.4 US 24, Leadville

A chip seal was constructed on US 24 south of Leadville, CO on July 11, 2006 by CDOT maintenance personnel. Three 500-foot evaluation test sections were located within the project in the northbound driving lane as shown on Figure 10. A control section was located as shown with no chip seal applied, a section was placed with no fog seal applied over the seal, and a section was placed with a fog seal applied over the completed chip seal.

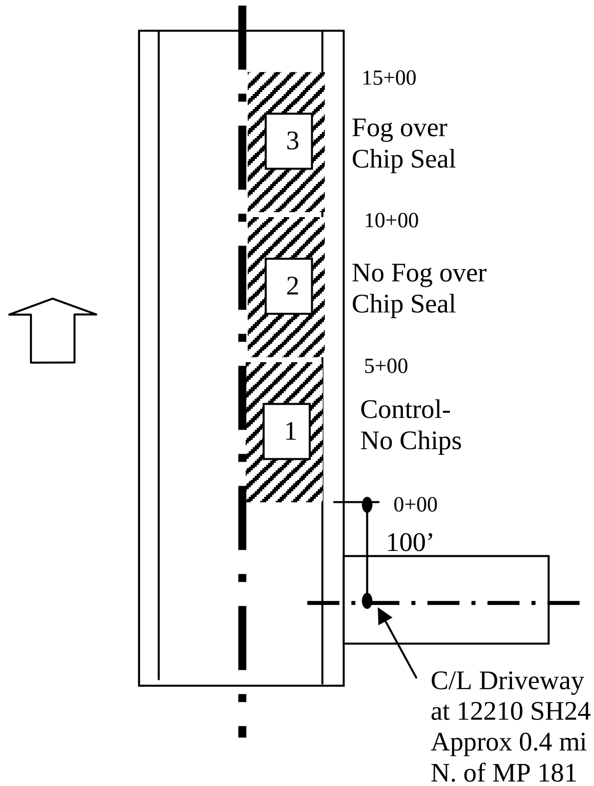


Figure 10. Chip Seal Test Sections on US 24, Leadville

3.3.5 US 50, Swink

A chip seal was constructed on US 50 east of Swink, CO on July 24, 2006 by CDOT maintenance personnel. Eight evaluation test sections were located within the project in both the eastbound driving and passing lanes on each side of Timpas Creek as shown on Figure 11.

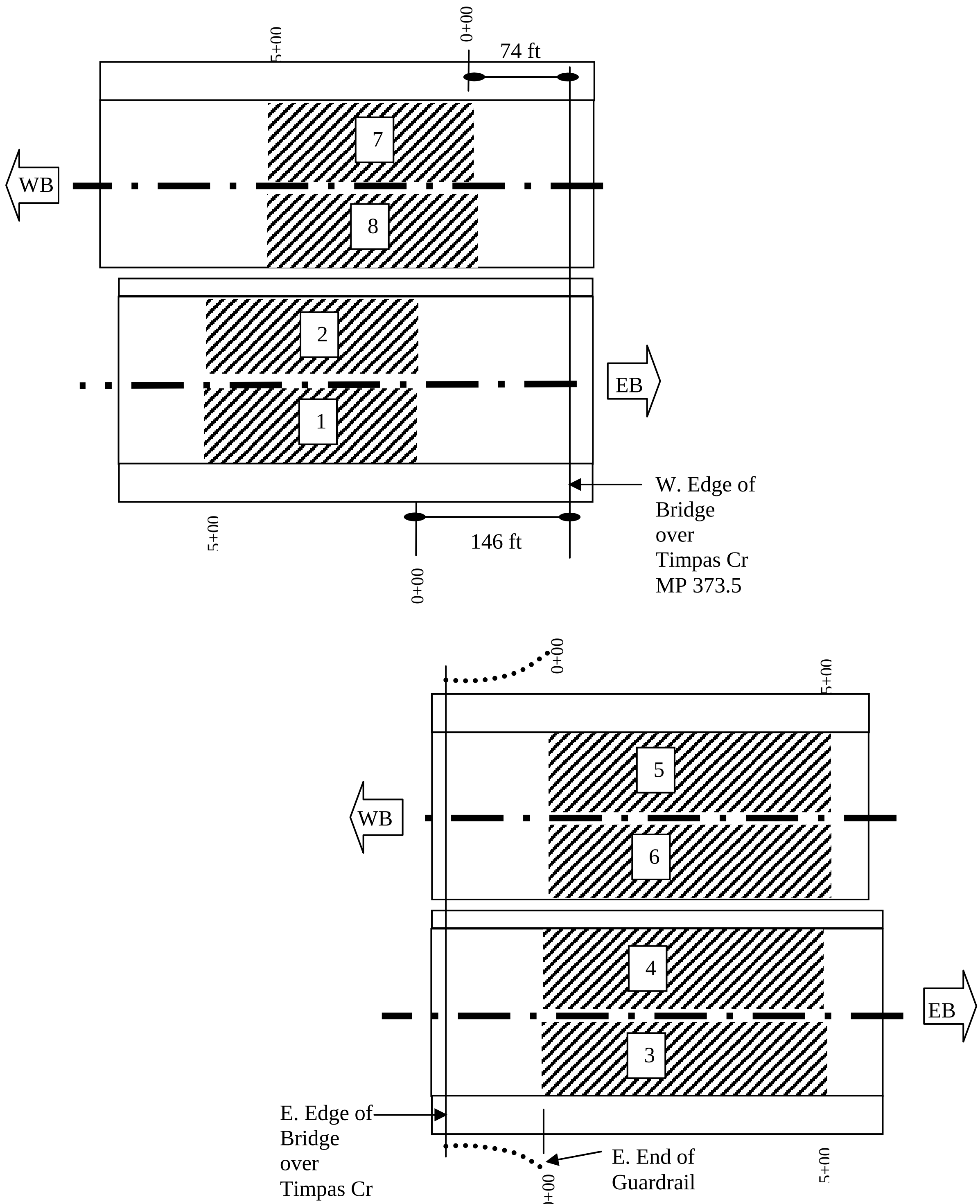


Figure 11. Chip Seal Test Sections on US 50, Swink

3.4 Thin Overlays

3.4.1 US 6, Golden

A thin hot mix asphalt (HMA) overlay was placed by CDOT forces on June 23, 2005 on US 6 in the southbound lanes at approximately 0.3 miles south of Milepost 273. Two 500-foot test sections were identified as shown in Figure 12.

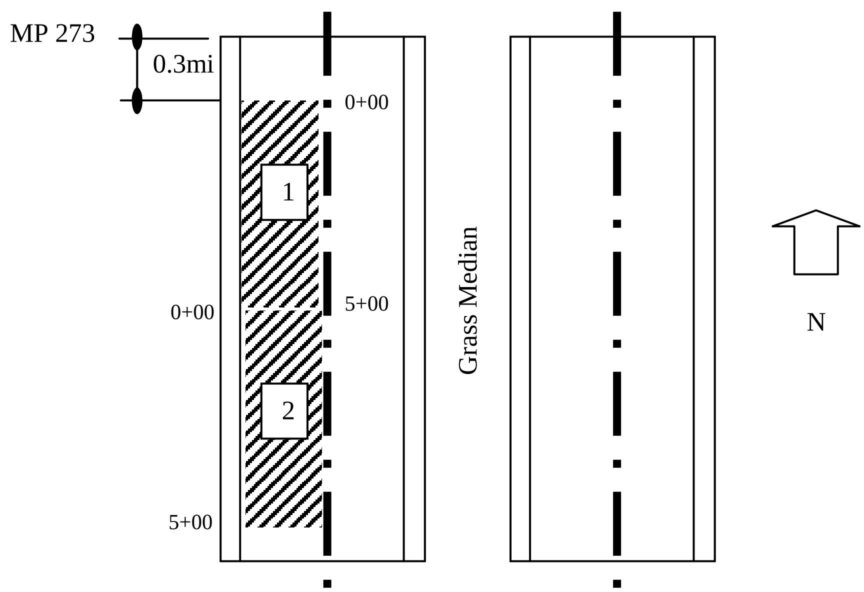


Figure 12. Thin Overlay Test Sections on US 6, Golden

3.4.2 SH121, Littleton

A thin hot mix asphalt overlay was placed by CDOT forces on May 16, 2006 on SH 121. Two 500-foot test sections were established in the southbound driving lane approximately 0.52 miles and 2.4 south of the centerline of Trailmark Parkway, respectively. No figure is included for this test section because the pavement was overlaid one year after the test sections were established so only one year of performance information is available.

3.4.3 US 550, Durango

The existing pavement was rehabilitated using hot in-place recycling (HIPR) and thin overlays in June and July, 2006. Evaluation test sections were established to evaluate the effects of utilizing HIPR with 1.5 and 2 inch overlays. A total of thirty test sections were established at this site as shown on Figure 13.

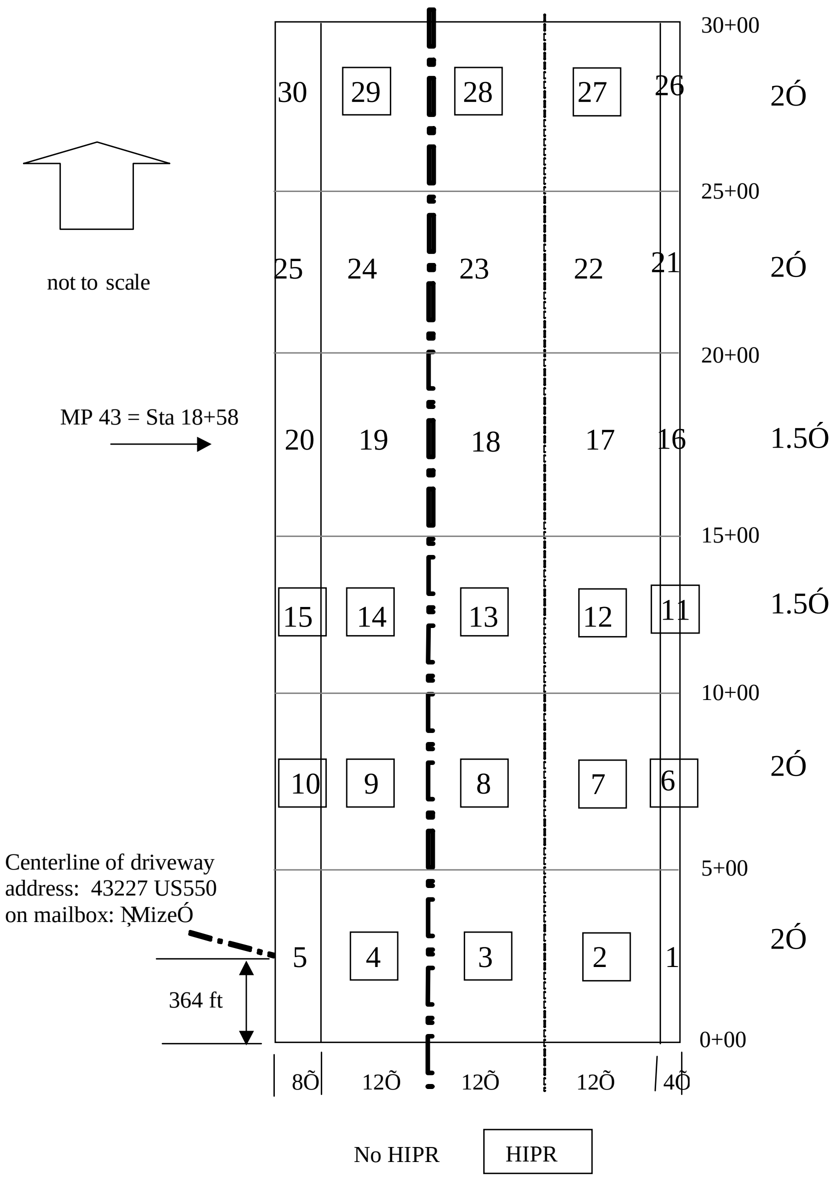


Figure 13. Thin Overlay Test Sections on US 550, Durango

3.4.4 US 550, Coal Bank Pass

A hot mix asphalt overlay was placed on May 30, 2006 on US550 south of Coal Bank Pass. Four 500-foot test sections were established in the northbound and southbound lanes approximately 100 feet south of Milepost 56 as shown on Figure 14. Evaluation sections 1 and 4 were placed 1.5 inches thick and sections 2 and 3 were placed 2 inches thick.

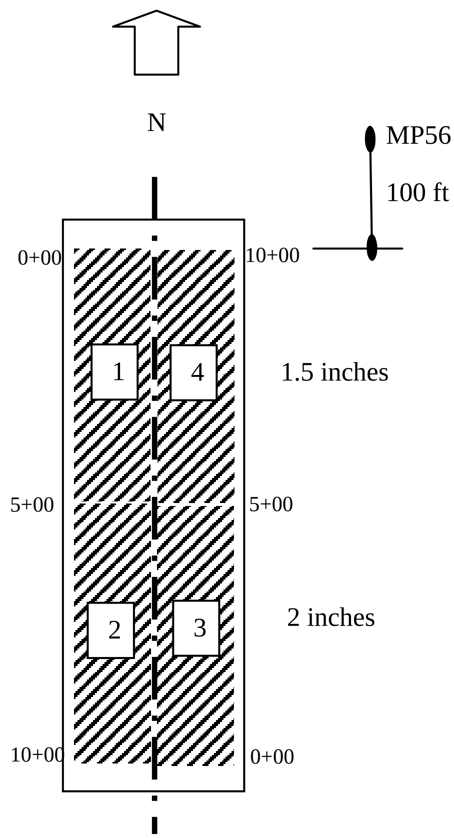


Figure 14. Thin Overlay Test Sections on US550, Coal Bank Pass

3.4.5 US 40, Golden

A 1-inch thick hot mix asphalt overlay was placed by CDOT forces on May 3, 2006 on US 40 in the eastbound lane at approximately Milepost 282 and approximately the centerline of the intersection with Mother Cabrini Shrine Road. Two 500-foot test sections were identified as shown in Figure 15.

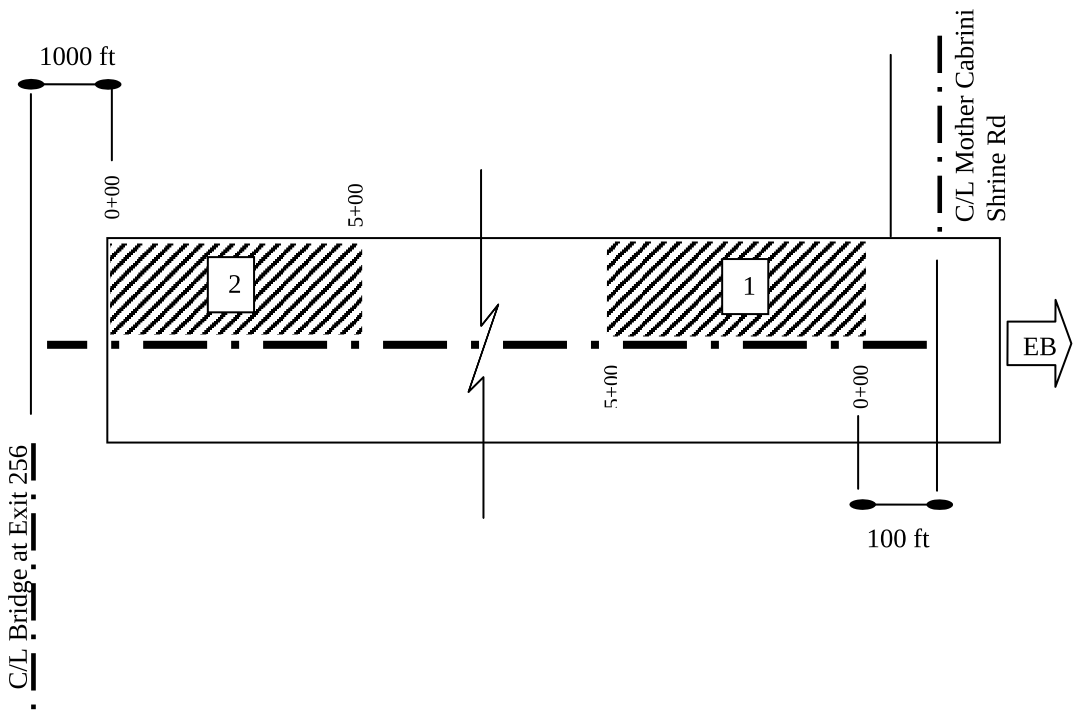
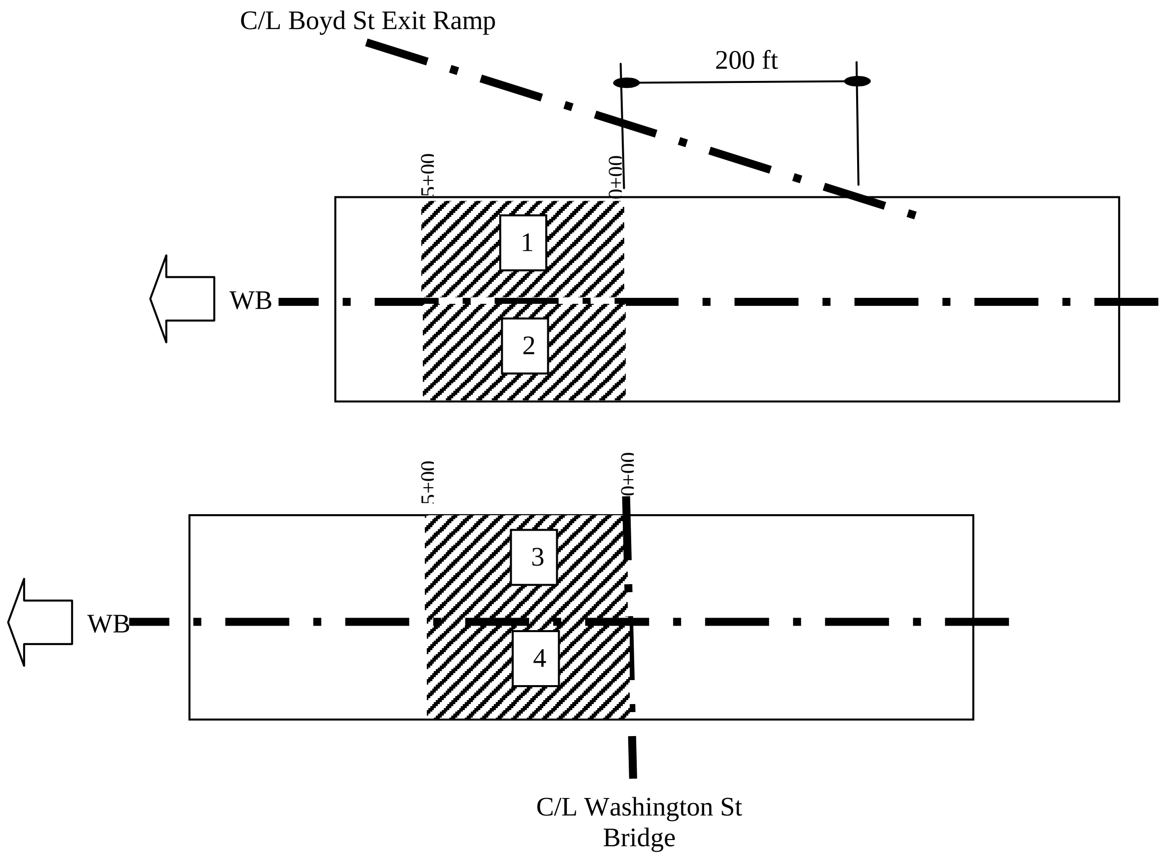


Figure 15. Thin Overlay Test Sections on US 40, Golden

3.5 Ultra-thin Bonded Wearing Course

3.5.1 SH 58, Golden

An ultra-thin bonded wearing course (UBWC) was placed by Lafarge in 2004 on SH 58 in Golden. Two test sections were established in the southbound driving lanes as shown in Figure 16 after construction on July 7, 2005 by measuring the cracking in the existing pavement.



3.5.2 Table Mesa Drive, Boulder

An ultra-thin bonded wearing course (UBWC) was placed by Lafarge in 2004 on Table Mesa Drive in Boulder. Two test sections were established in the eastbound driving lanes as shown in Figure 17 after construction on June 8, 2005 by measuring the concrete joint reflection cracking in the pavement surface.

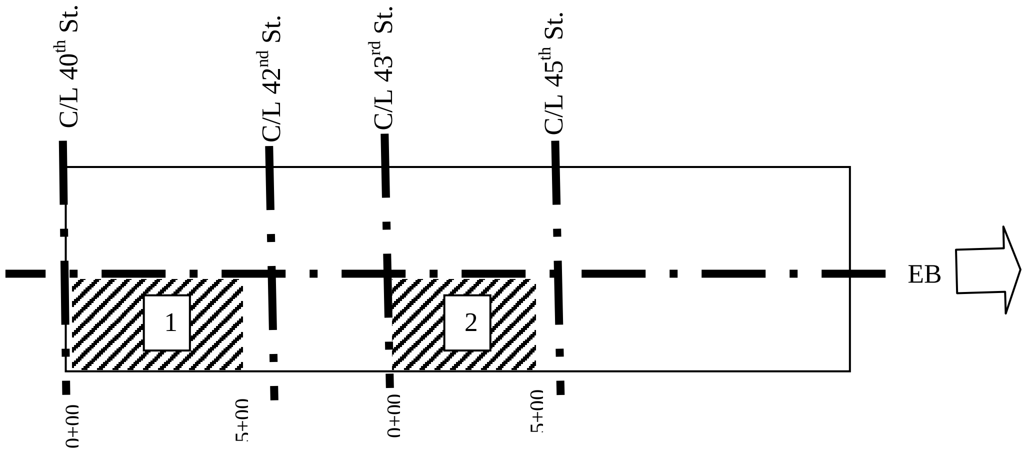


Figure 17. UBWC Test Sections on Table Mesa Dr., Boulder

3.6 Concrete Joint Resealing

3.6.1 US 287, Campo

Joint resealing test sections were placed on the concrete pavement on US 287 approximately 2.6 miles south of Campo at Milepost 3 in September, 2005. Test sections consisted of removing existing sealant by sawcutting and replacing with new sealant. Control sections consisted of leaving the existing sealant in place and not resealing. The locations of these test sections are shown on Figure 18. The original plan was to place test and control sections in the southbound lane. However, the contractor mistakenly removed and replaced all the sealant in the southbound lane. Therefore, the two sections shown in Figure 18 in the northbound lane will be used as control sections where sealant was not removed and replaced.

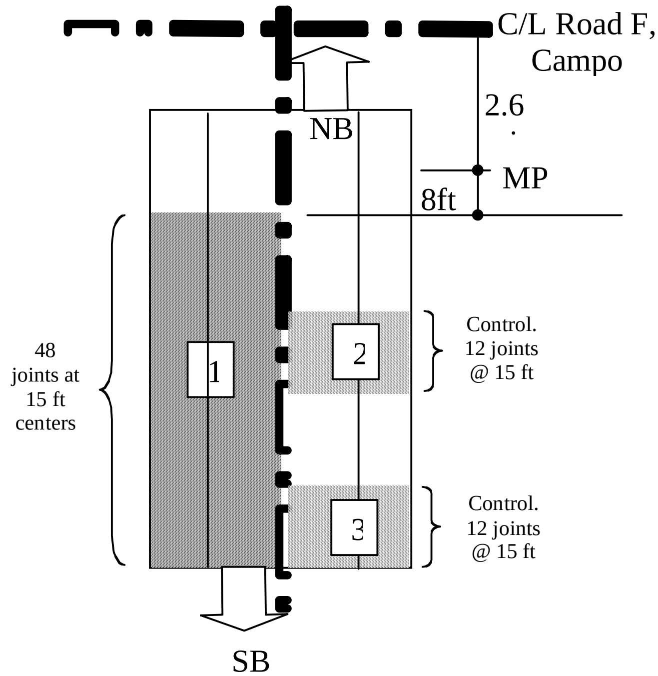


Figure 18. Joint Reseal Test Section Location on US 287, Campo

3.7 Concrete Cross-stitching

3.7.1 US287, Campo: Cross-stitching

Cross-stitching test sections were placed on the concrete pavement on US 287 approximately 1.0 miles south of Campo at Milepost 8 in September, 2005. Test sections consisted of repair of a longitudinal crack in the concrete pavement using both conventional deformed reinforcement cross-stitches and fibreglass panels manufactured by Uretek, Inc. as shown in Figure 19.

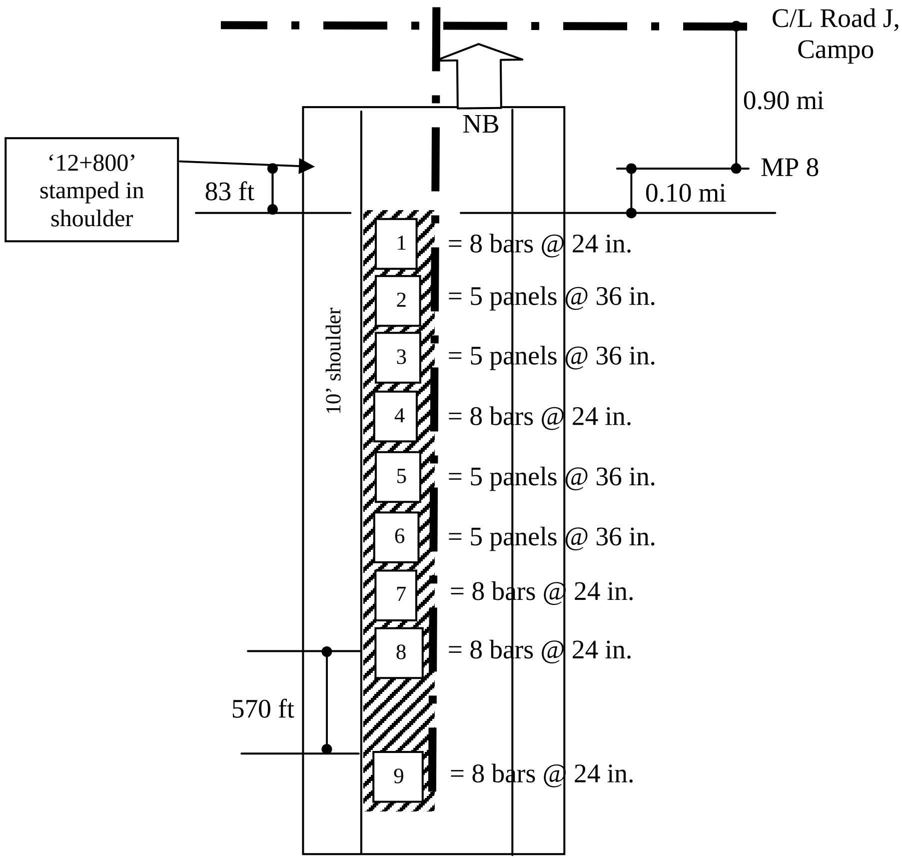


Figure 19. Cross-stitch Test Section Location on US 287, Campo

3.8 Concrete Pavement Diamond Grinding

3.8.1 I-70, Rifle

Diamond microgrinding of the concrete pavement on I-70 near Rifle was done by American Civil Constructors, Inc. in September, 2005. Test and control sections to evaluate the effectiveness of this technique at approximately Milepost 96 in the eastbound driving lane were installed on September 13, 2005. No grinding of the pavement surface occurred in the control sections. Locations of the test and control sections are shown in Figure 20.

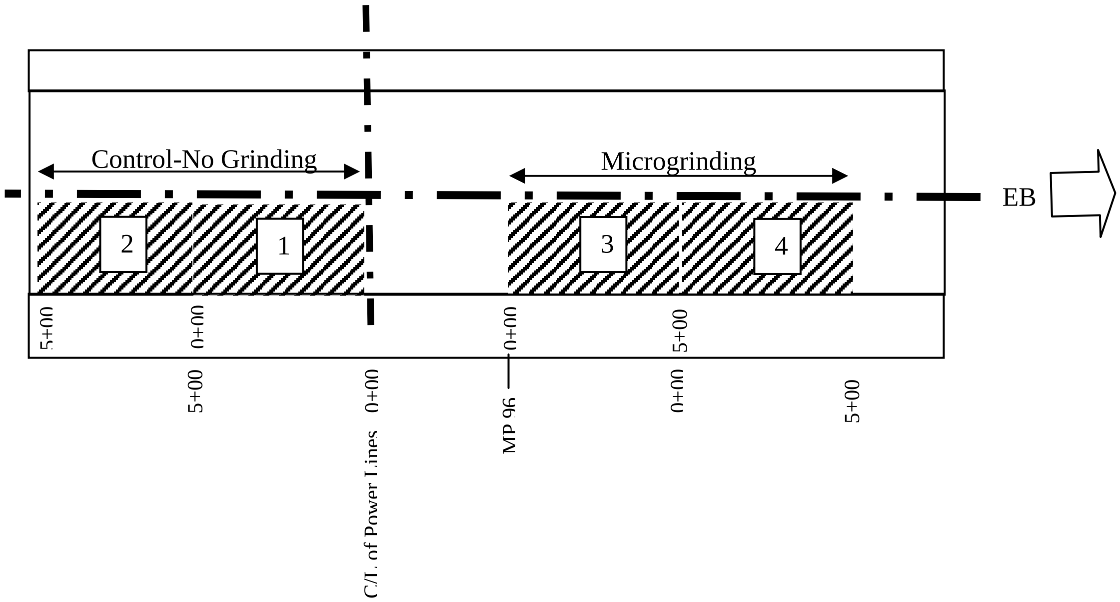


Figure 20. Microgrinding Test Section Location on I-70, Rifle

4.0 TEST SECTION PERFORMANCE

Visual condition surveys were conducted annually for each test location. Condition surveys followed the protocol outlined by the Federal Highway Administration (Miller and Bellinger 2003). Evaluation sections were each 500 feet in length divided into five 100 segments. Distress was recorded within each of these five segments and an average obtained. Figures in the following sections show distress for each test section on a specific pavement. Each line on the figure represents one test section and the bold black line is the average of the representative sections. The ordinate axis on the graphs is the average length of longitudinal and transverse cracking or area of alligator cracking, rutting or raveling observed for the 100 foot segments within each evaluation section.

4.1 Crack Seal

Performance of the crack sealants was evaluated for four years by comparing the length of crack unfilled to the length of crack filled. The Crack Fill Integrity shown on Figures 21 and 22 over time is a measurement of the length of crack fill remaining intact over time. Crack Fill Integrity, % = 100 (Unfilled Crack Length/Original Filled Crack Length).

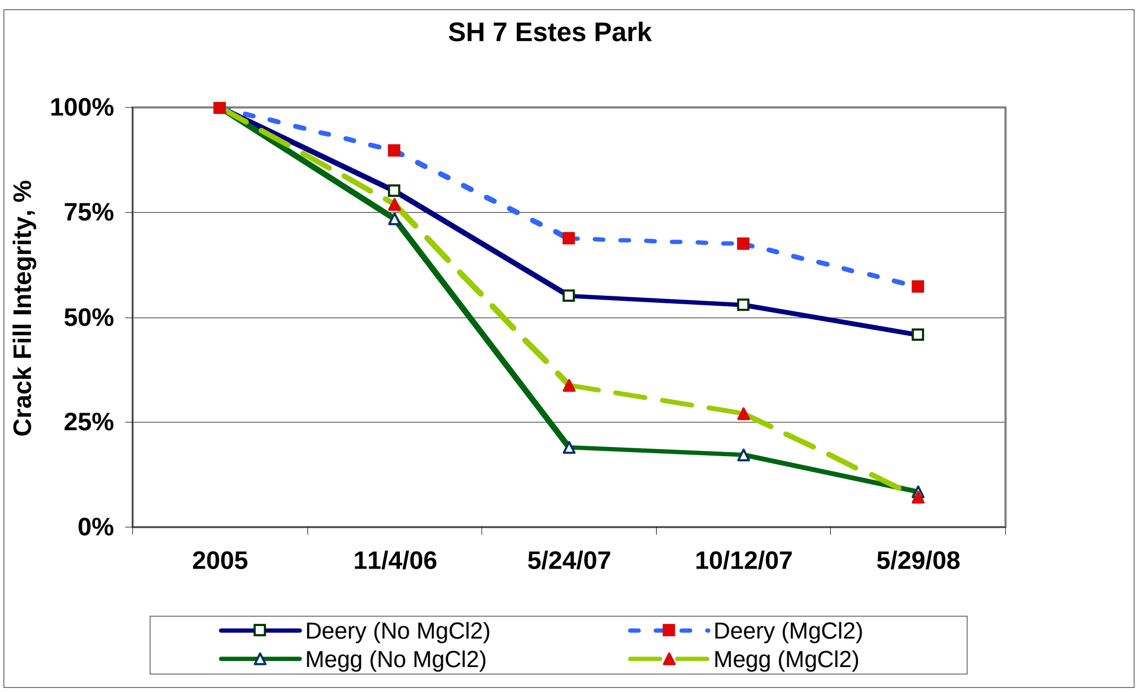


Figure 21. Crack Fill Integrity, SH 7, Estes Park

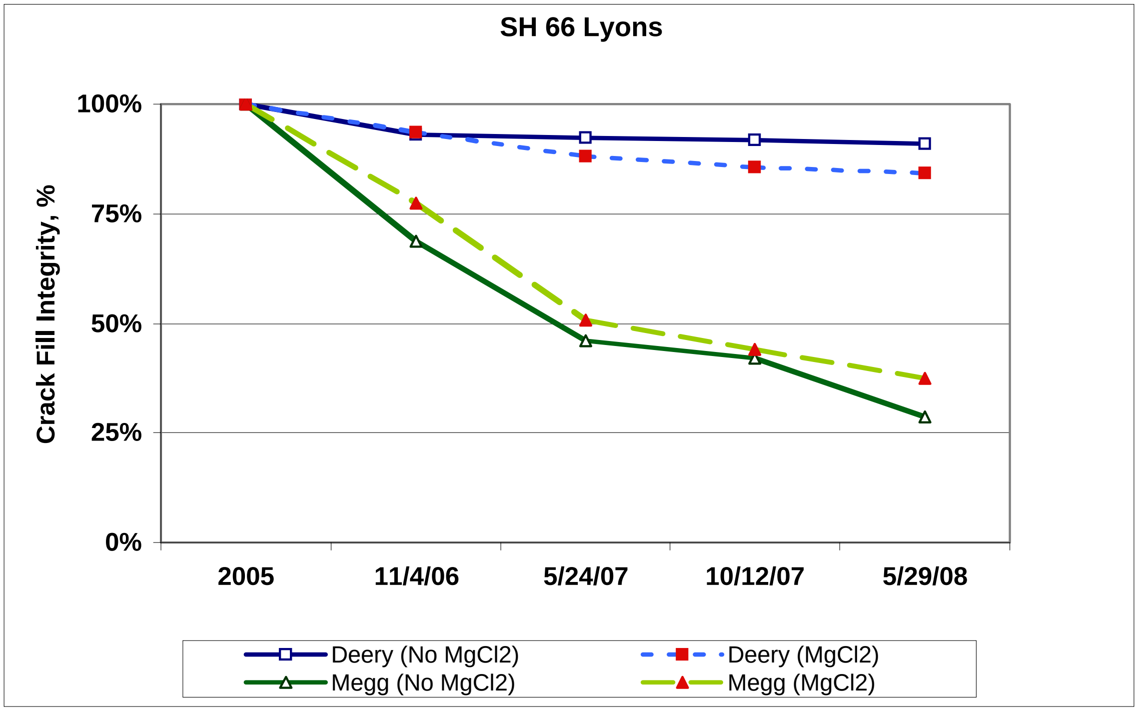


Figure 22. Percent Crack Fill Remaining, SH 66, Lyons

4.2 Thin SMA

4.2.1 SH 74, Evergreen

Construction of the SMA on SH 74 occurred one year prior to the beginning of this research. However, the project was included in this study to provide some indication of thin SMA performance. Pavement condition data collection began one year after construction as shown on Figures 23 and 24 for the longitudinal and transverse cracking that began appearing after one year. Results shown in Figures 23 and 24 indicate each of two test sections (solid and dashed lines) and the average (bold solid).

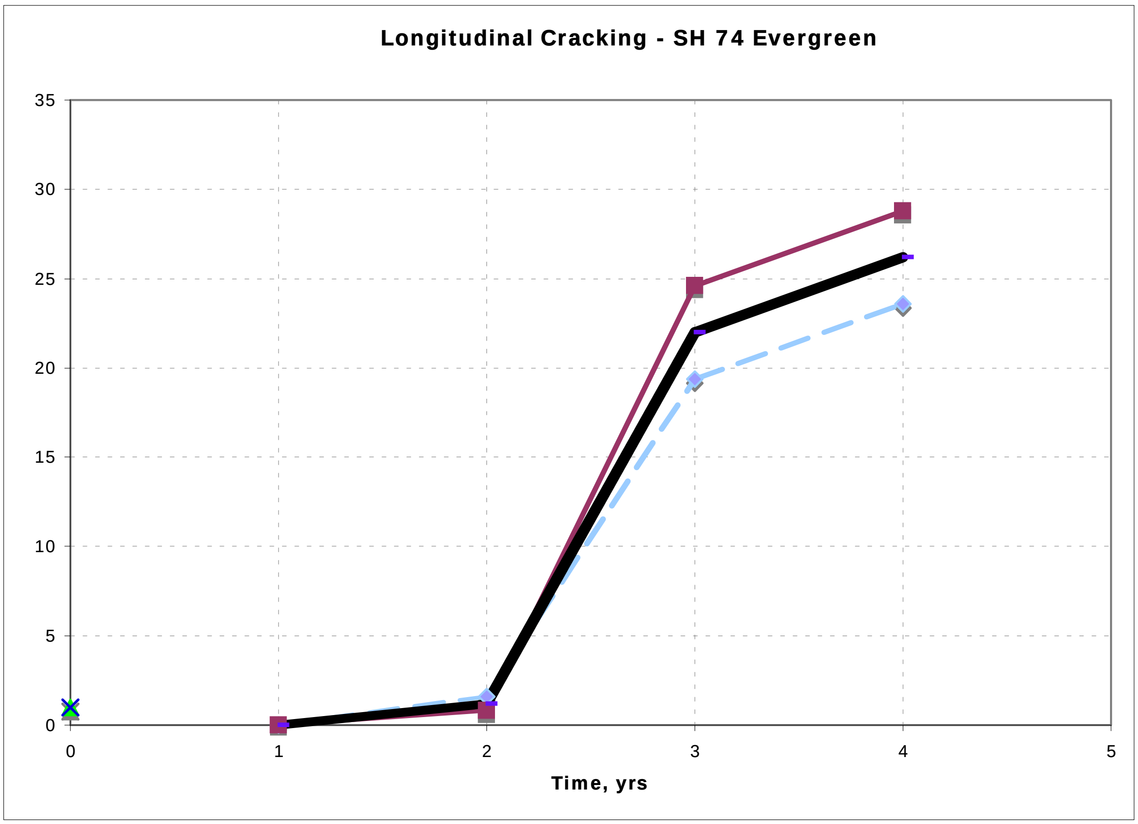


Figure 23. Longitudinal Cracking/500 ft Section for Thin SMA on SH 74, Evergreen

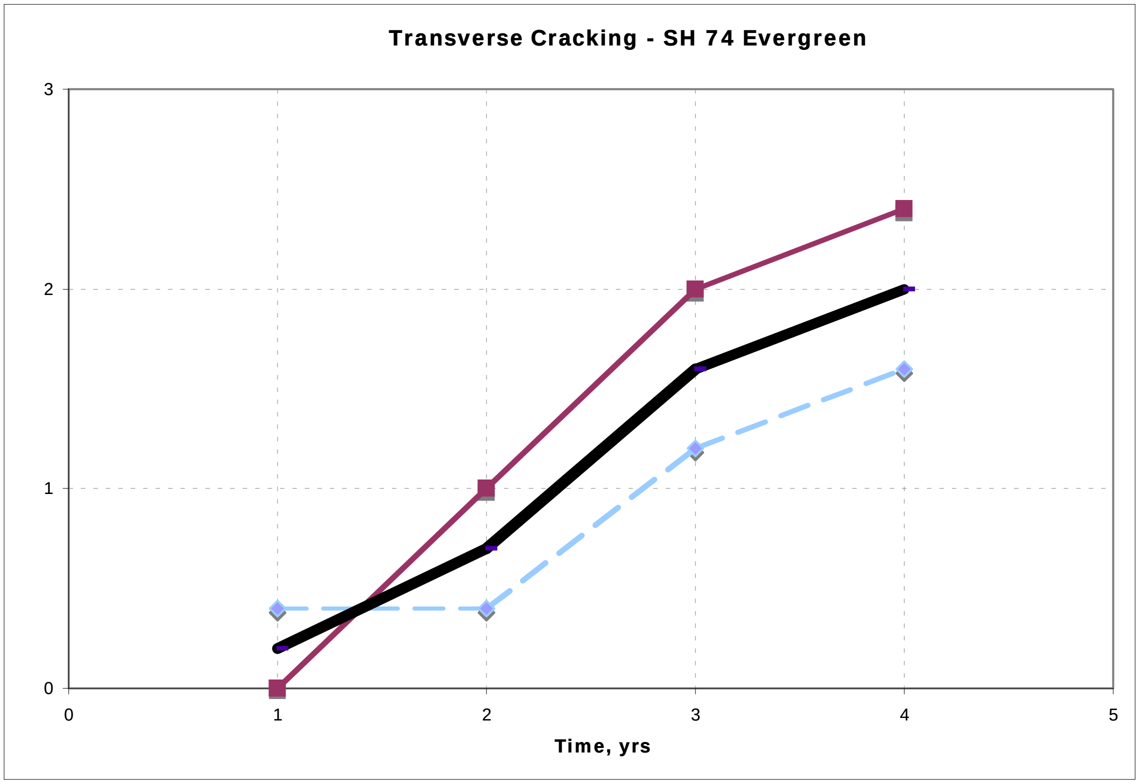


Figure 24. Transverse Cracking /500 ft Section for SH 74, Evergreen

4.2.2 SH 13, Rifle

The Rifle test sections include driving and passing lanes of the northbound SH 13 by-pass. Two evaluation sections for each lane were monitored and the results are shown on Figures 25 through 28. Average condition is shown as the solid bold line on each figure.

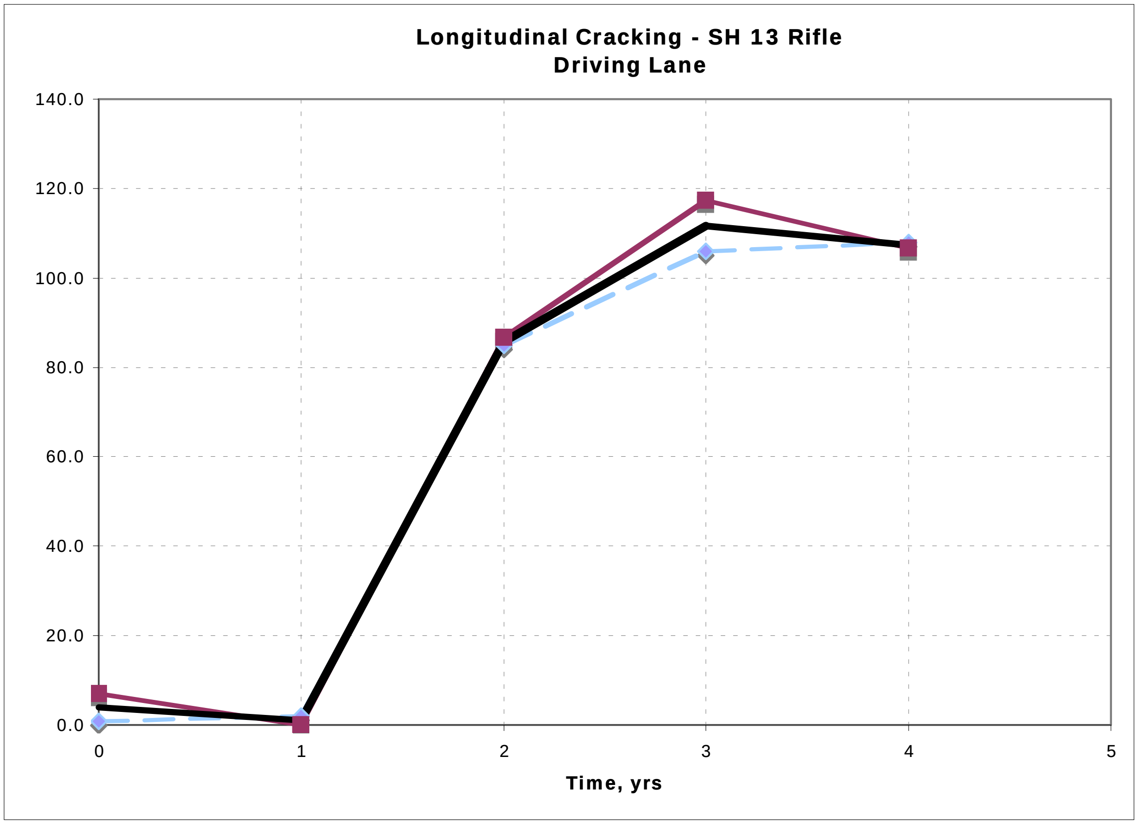


Figure 25. Longitudinal Cracking /500 ft Section for Thin SMA on SH 13, Rifle

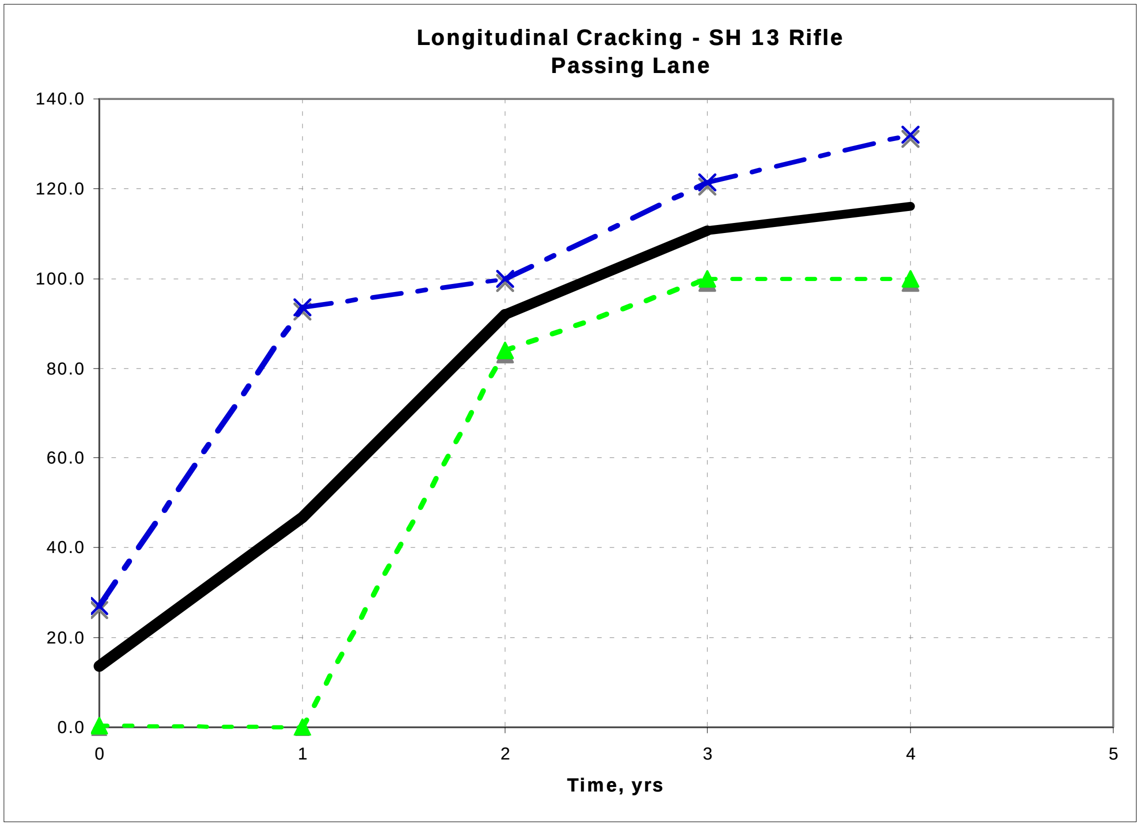


Figure 26. Longitudinal Cracking /500 ft Section for Thin SMA on SH 13, Rifle

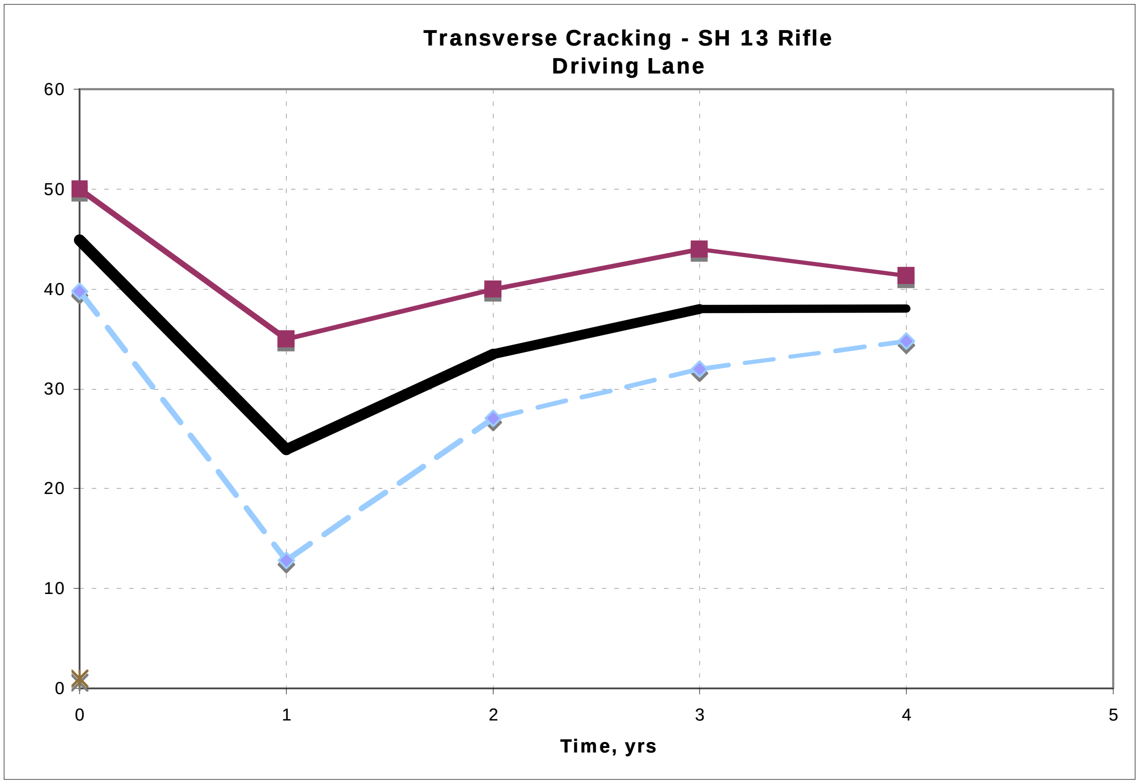


Figure 27. Transverse Cracking /500 ft Section for Thin SMA on SH 13, Rifle

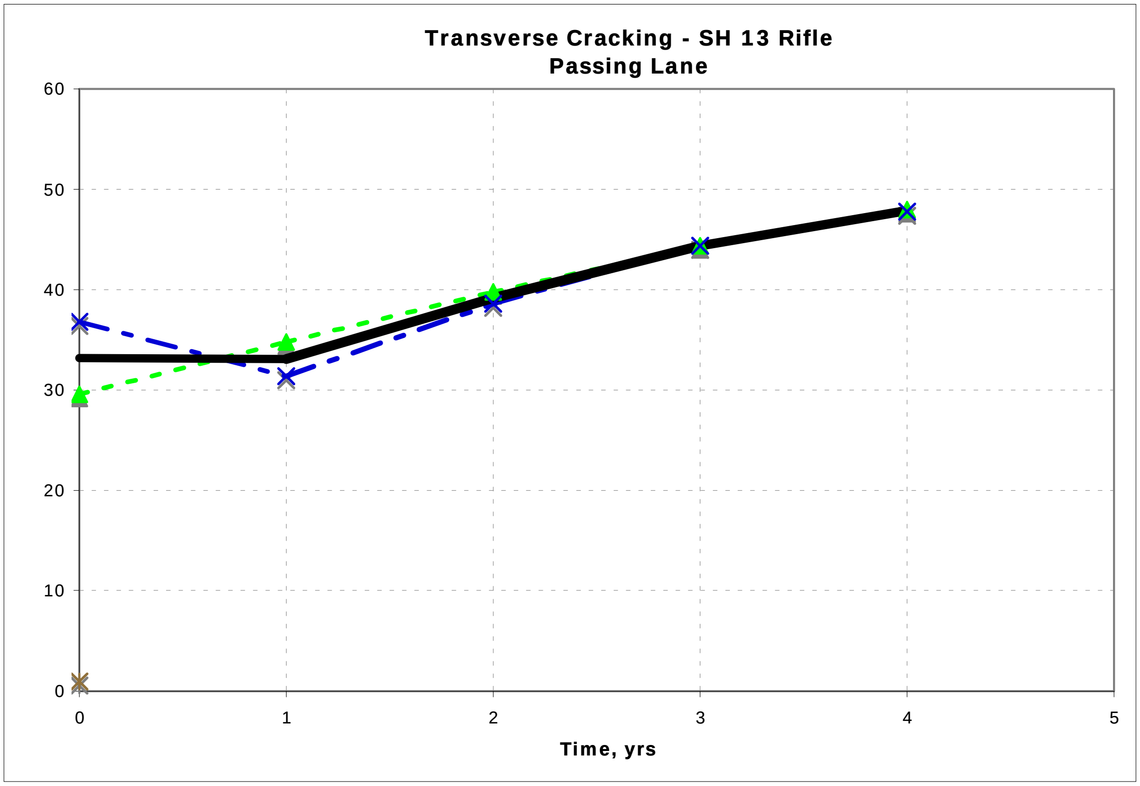


Figure 28. Transverse Cracking /500 ft Section for Thin SMA on SH 13, Rifle

4.2.3 I-70, Hanging Lake

The Hanging Lake test sections include the driving lanes of eastbound and westbound I-70. Two evaluation sections for each direction were monitored and the results are shown on Figures 29 through 32. Average condition is shown as the solid bold line on each figure.

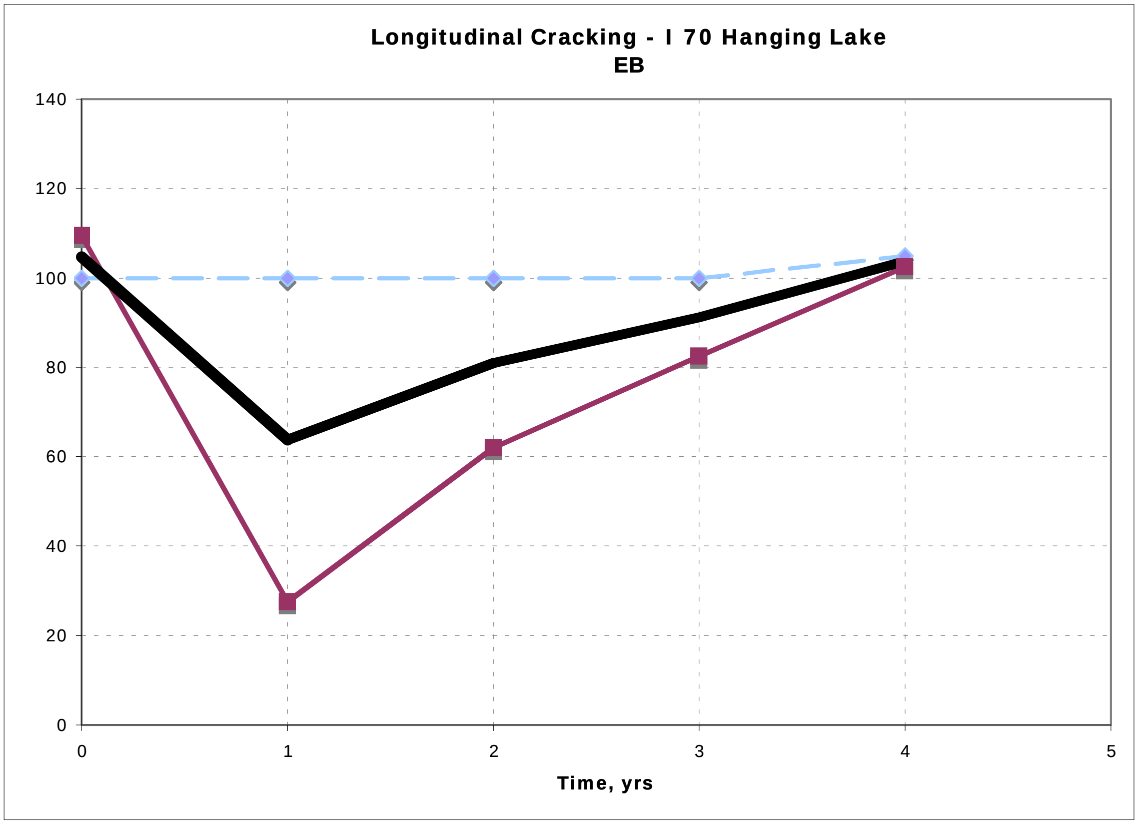


Figure 29. Longitudinal Cracking /500 ft Section for Thin SMA on I-70, Hanging Lake

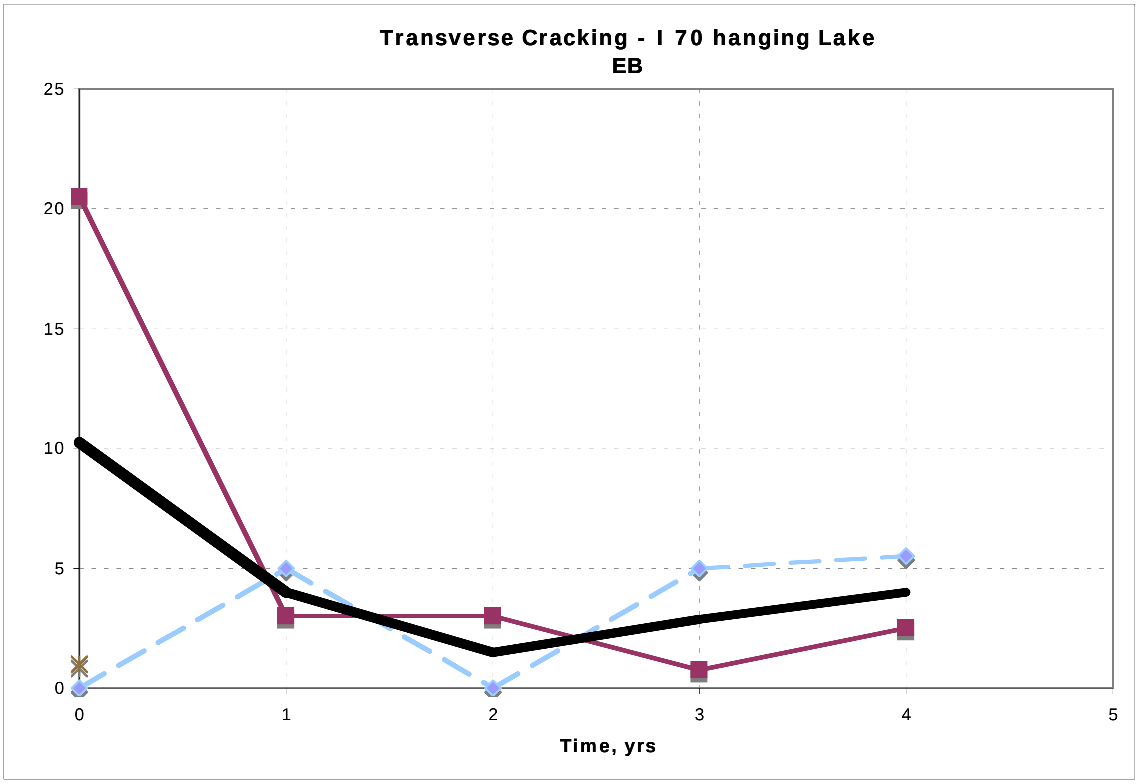


Figure 30. Transverse Cracking /500 ft Section for Thin SMA on I-70, Hanging Lake

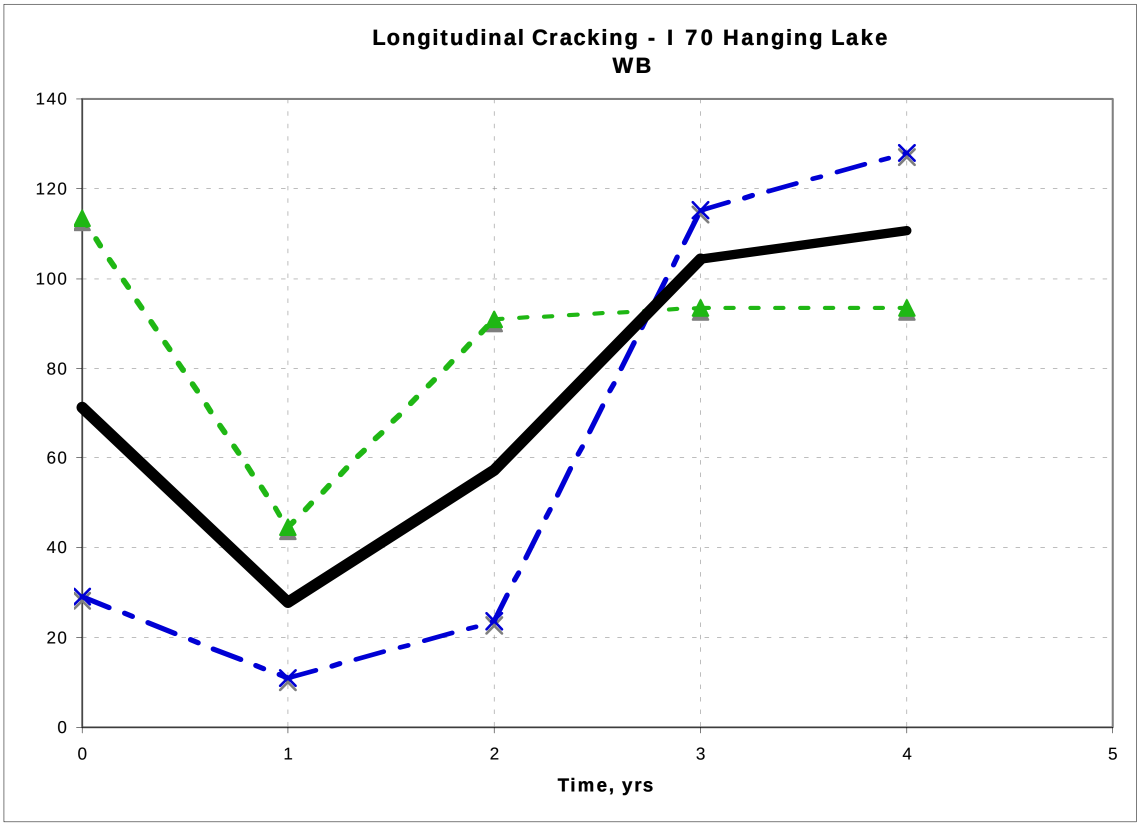


Figure 31. Longitudinal Cracking /500 ft Section for Thin SMA on I-70, Hanging Lake

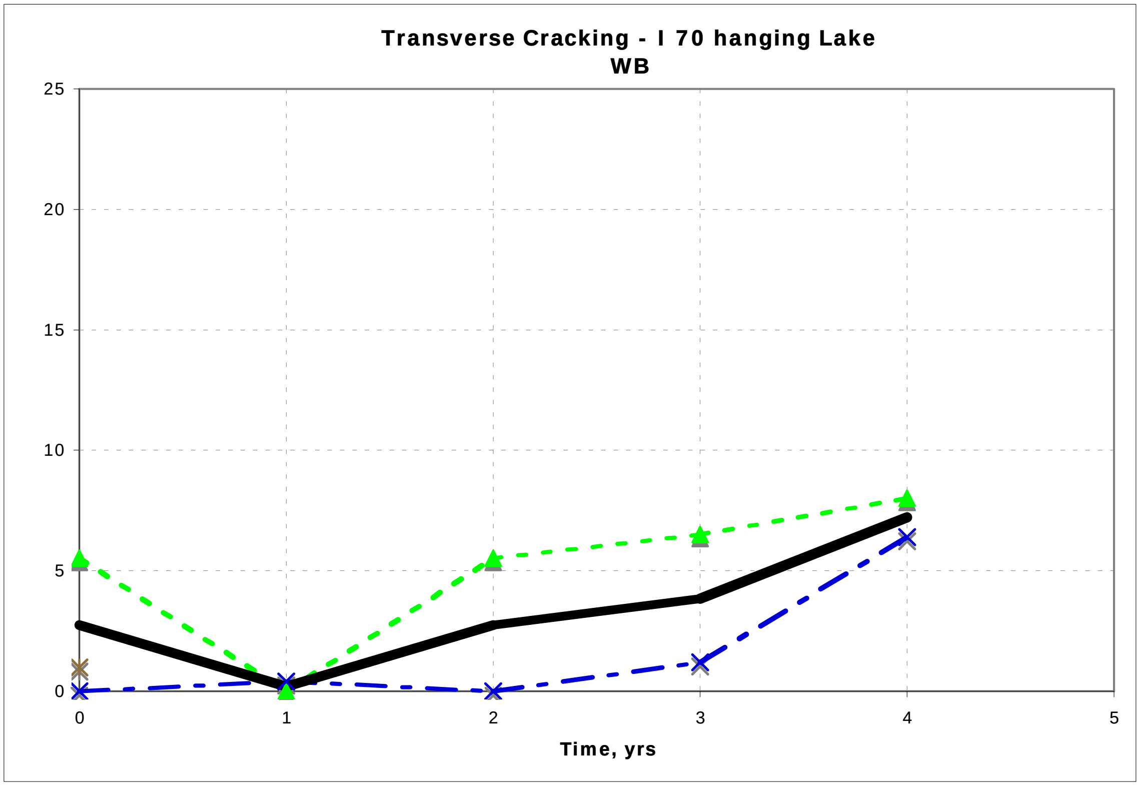


Figure 32. Transverse Cracking /500 ft Section for Thin SMA on I-70, Hanging Lake

4.3 Chip Seals

4.3.1 US 34, Drake

The test sections on US 34 at Drake include two evaluation sections in the eastbound and two sections in the westbound direction. Results of condition surveys are shown on Figures 33 through 36. Average condition is shown as the solid bold line on each figure.

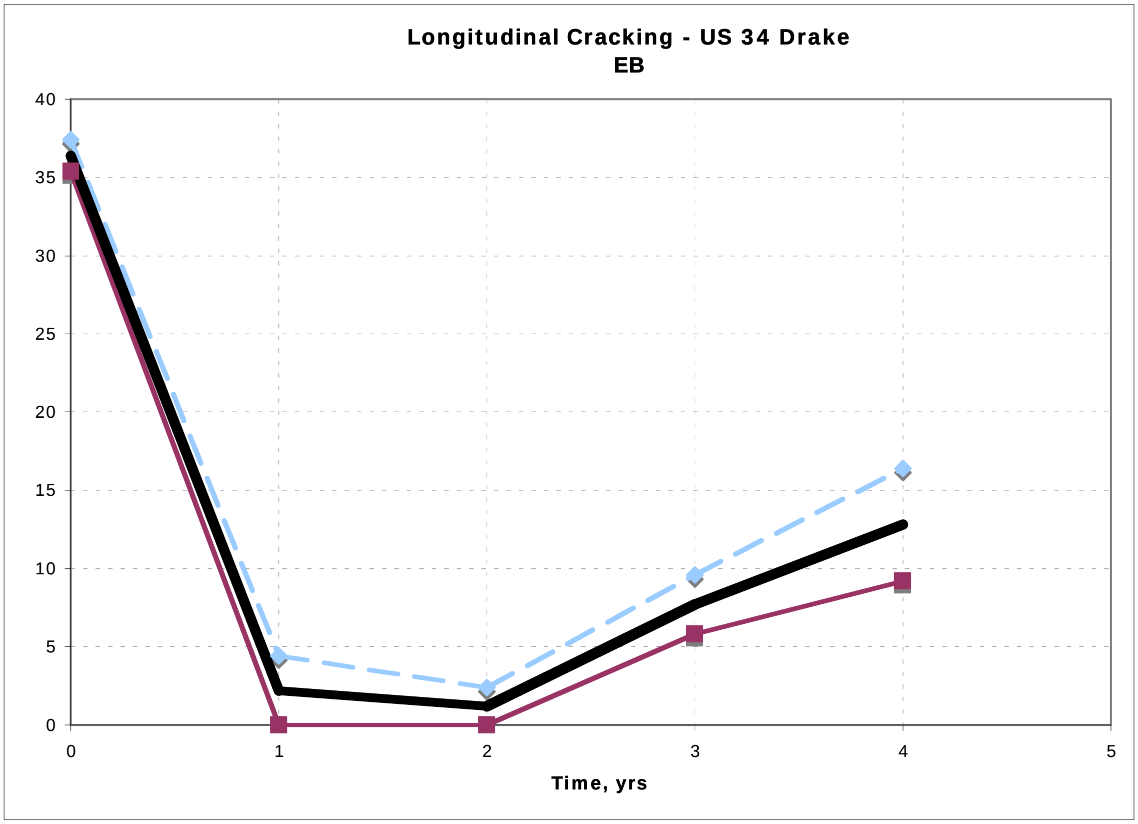


Figure 33. Longitudinal Cracking /500 ft Section for US 34, Drake

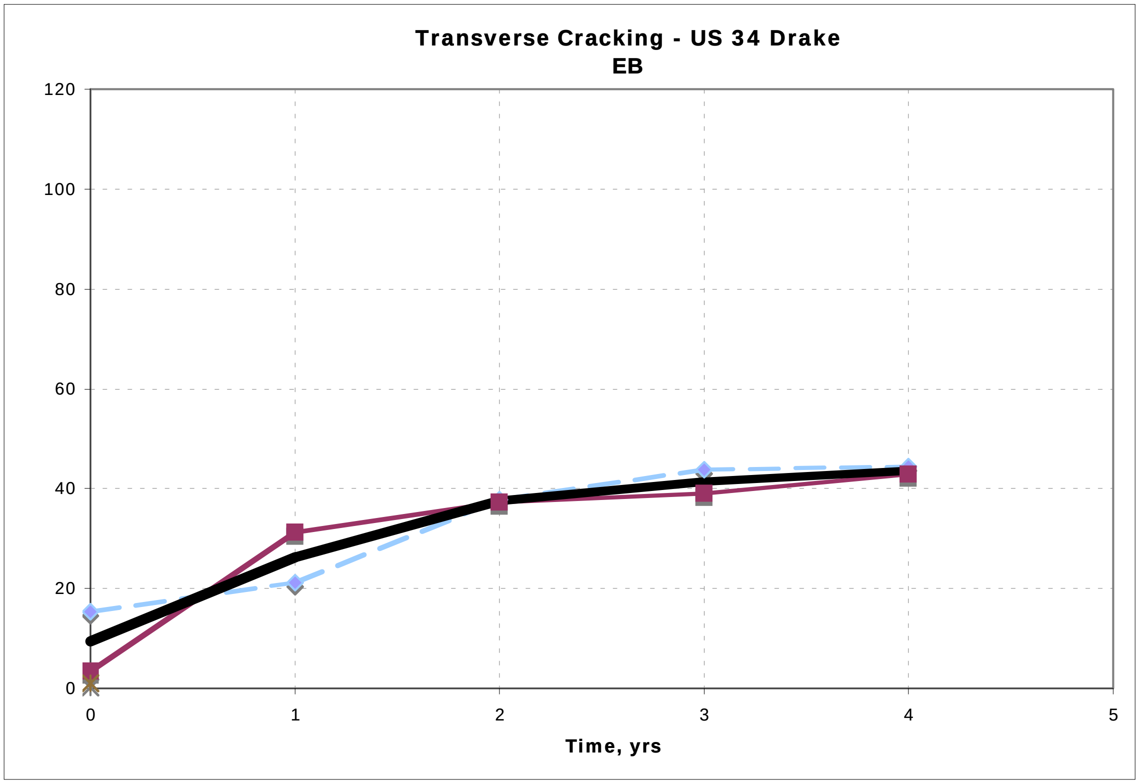


Figure 34. Transverse Cracking /500 ft Section for Chip Seal on US 34, Drake

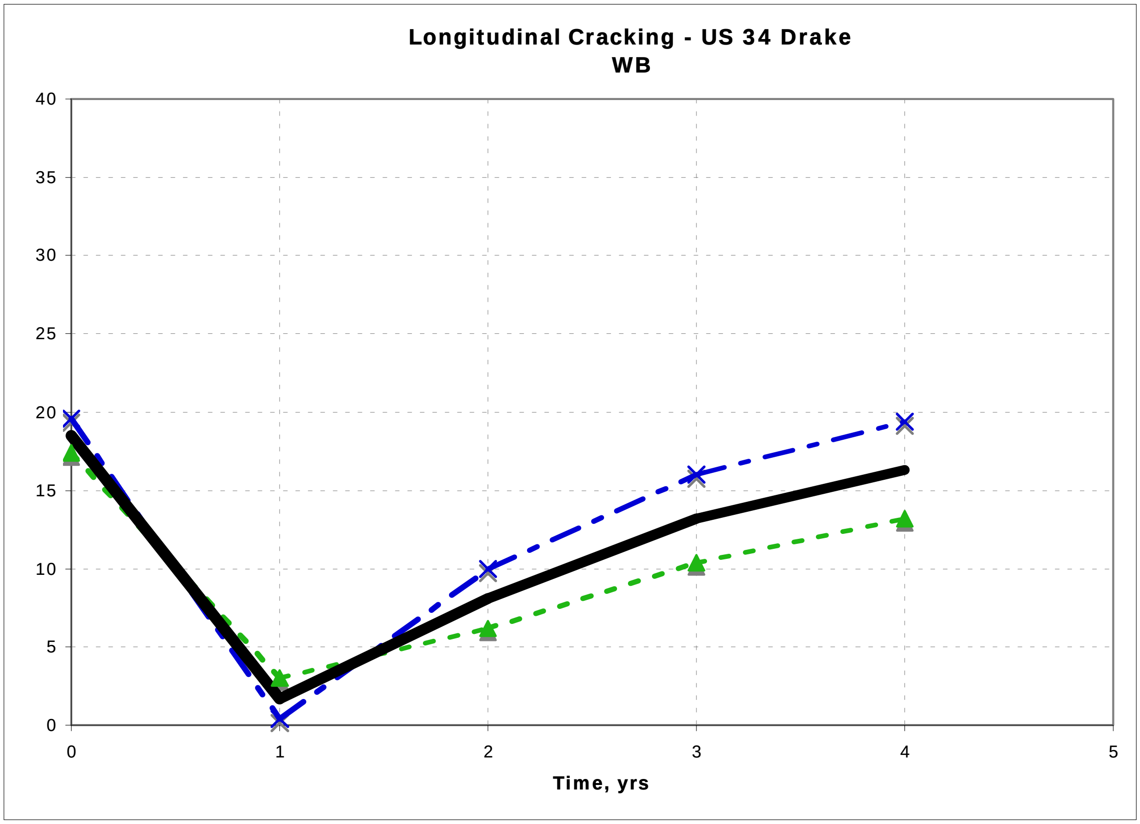


Figure 35. Longitudinal Cracking /500 ft Section for Chip Seal on US 34, Drake

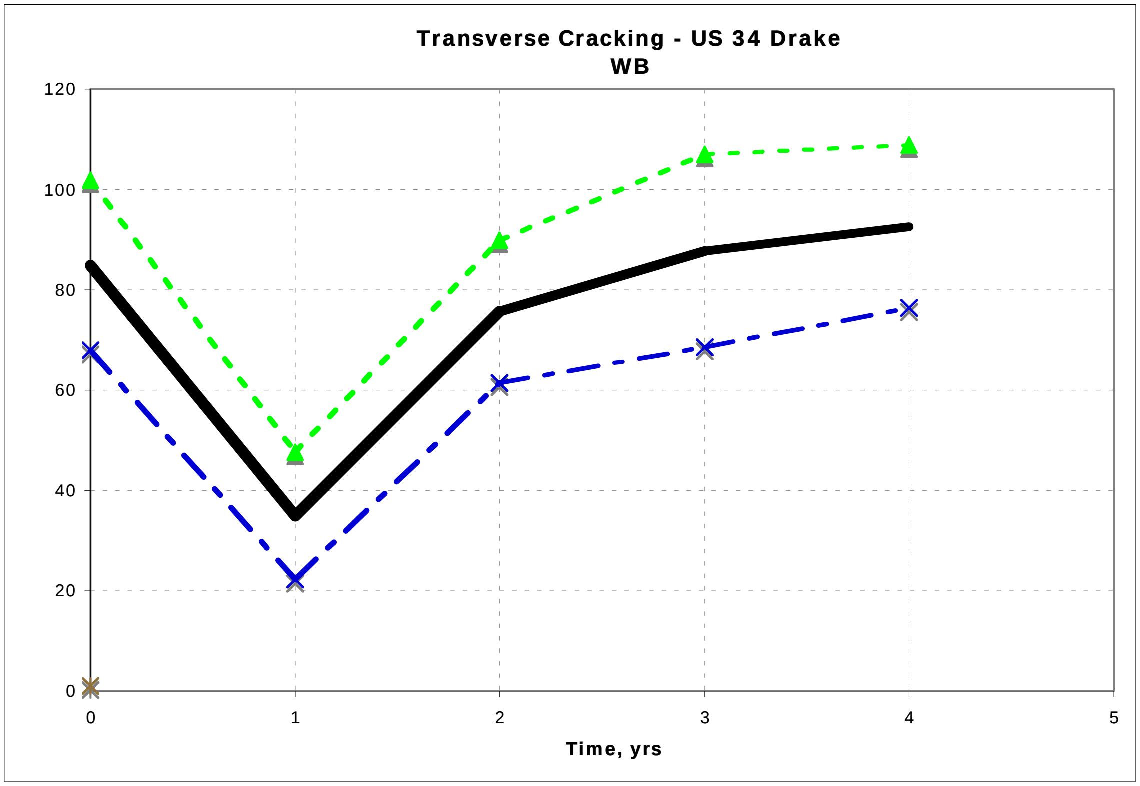


Figure 36. Transverse Cracking /500 ft Section for Chip Seal on US 34, Drake

4.3.2 SH 14 Briggsdale

The test sections on SH 14 west of Briggsdale include chip seals with two different chip gradations. Sections 1 and 2 were constructed with ½-inch chips and sections 3 and 4 were constructed using 3/8-inch chips. Results of condition surveys are shown on Figures 37 through 40. Average condition is shown as the solid bold line on each figure.

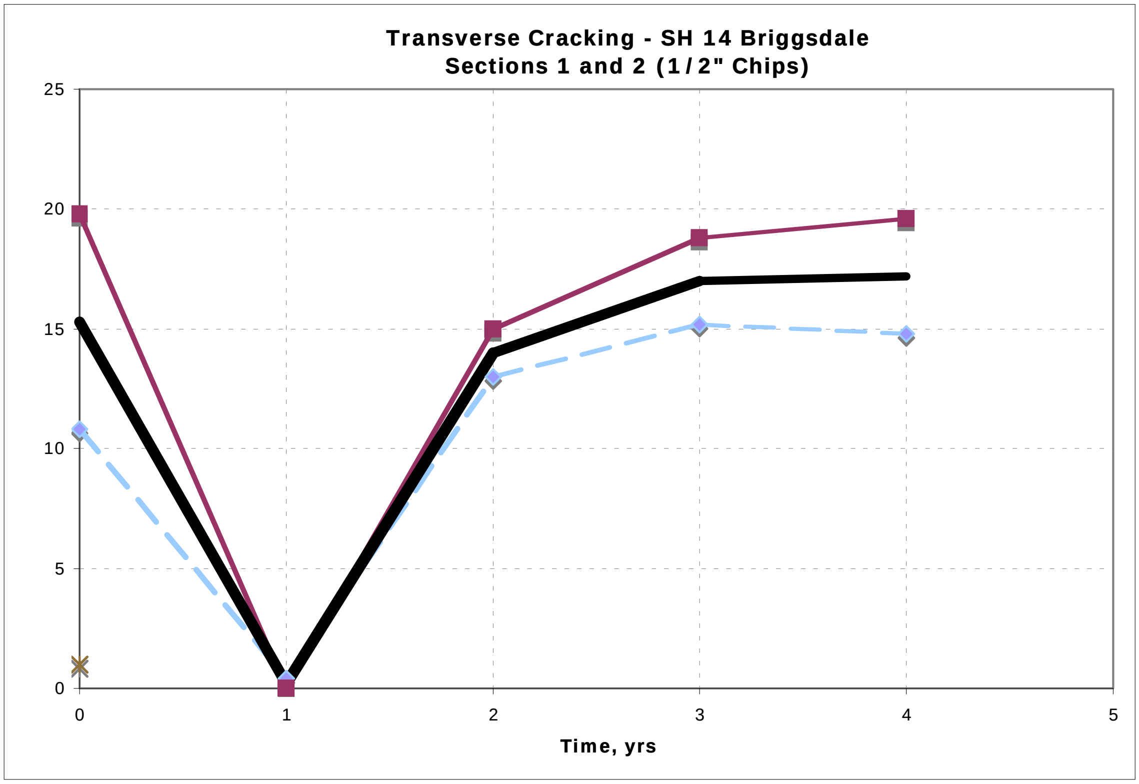


Figure 37. Transverse Cracking /500 ft Section for ½-inch Chip Sections on SH 14, Briggsdale

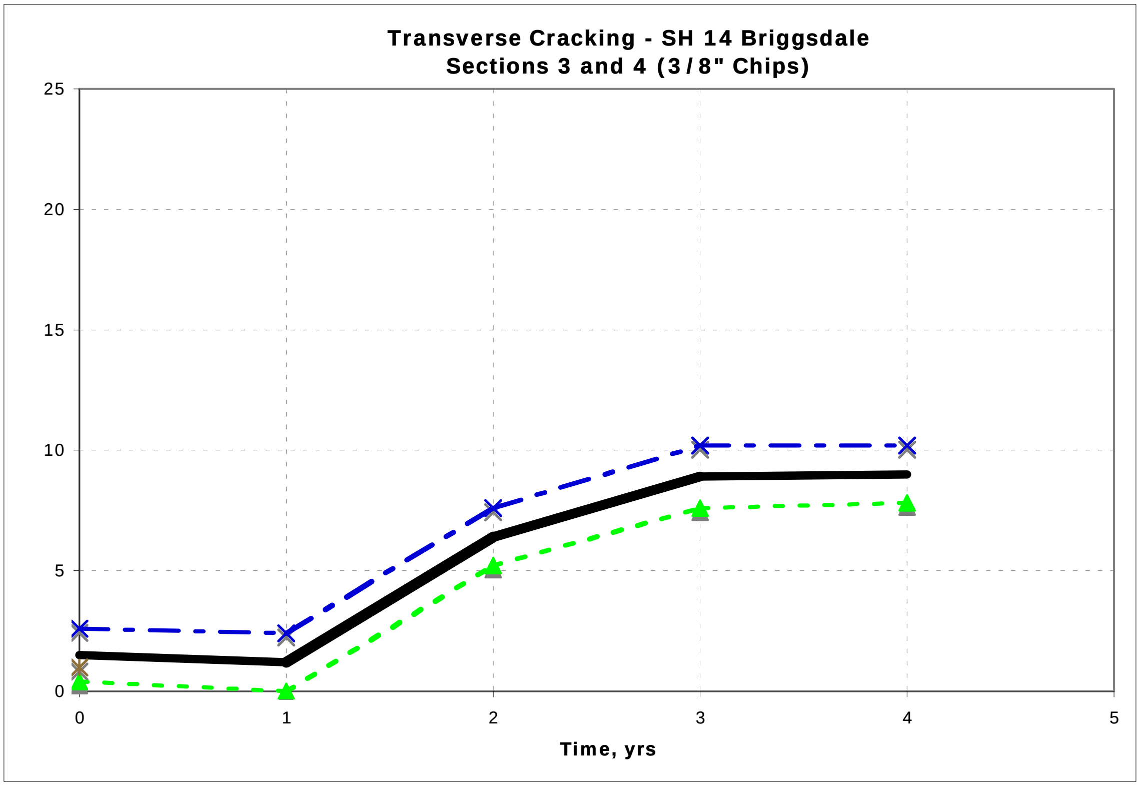


Figure 38. Transverse Cracking /500 ft Section for 3/8-inch Chip Sections on SH 14, Briggsdale

4.3.3 US 285, Poncha Pass/Springs

The test sections on US 285 near Poncha Springs and on Poncha Pass include a total of twenty evaluation sections. Performance of these will be presented with respect to the three types of treatments evaluated. These treatments are a control section with no chip seal, a chip seal with no fog seal applied, and a chip seal with a fog seal applied. These treatments were applied at the top of Poncha Pass and below the pass at the Arkansas River near Poncha Springs. Performance for these test sections is presented separately for the "Pass" and the "Springs" because of the potential differences in performance. Average values shown in the figures are for each of two control sections, two no fog seal sections and six fog sealed sections. Results of these condition surveys are shown on Figures 39 through 50. Average condition is shown as the solid bold line on each figure.

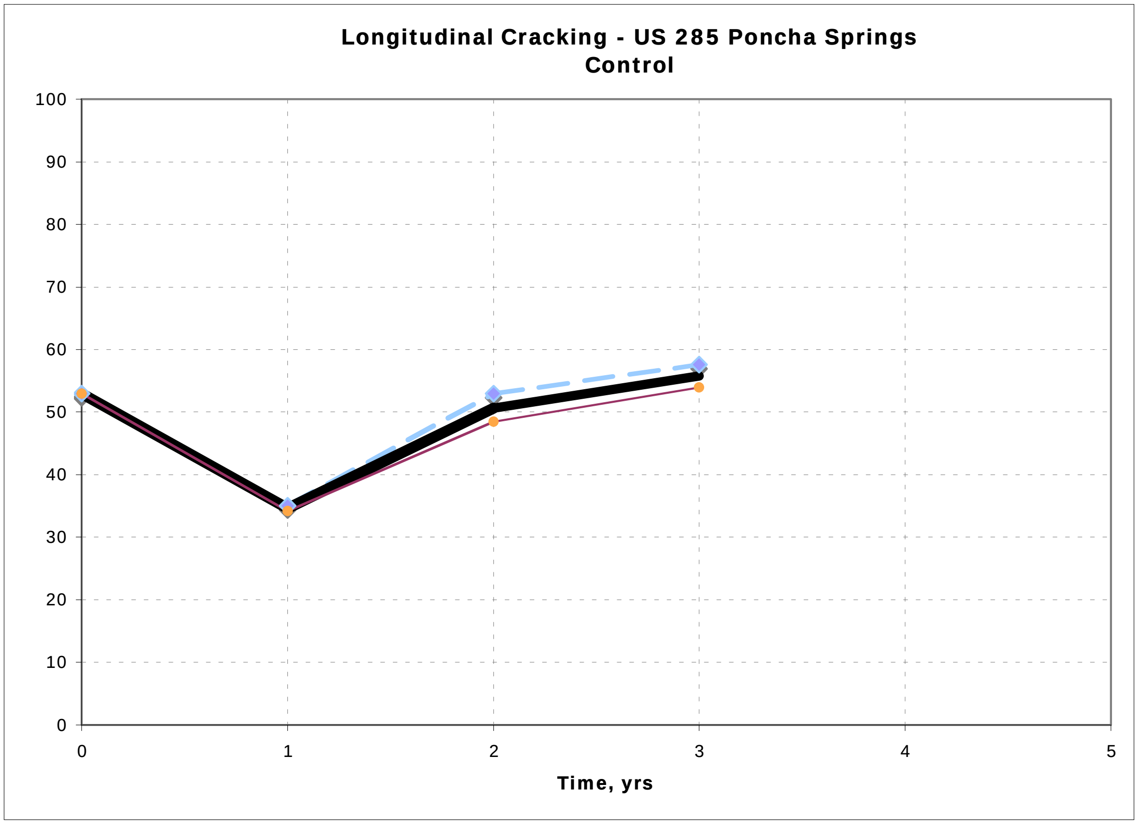


Figure 39. Longitudinal Cracking /500 ft Section for Control Sections on US 285, Poncha Springs

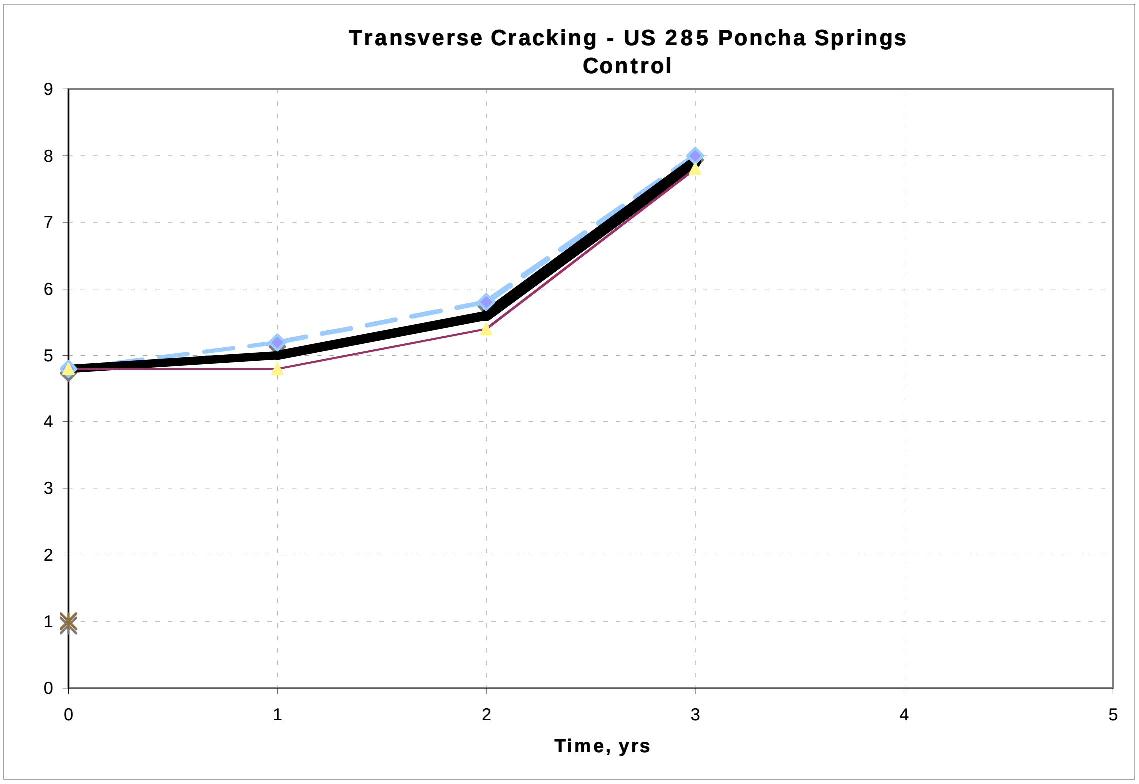


Figure 40. Transverse Cracking /500 ft Section for Control Sections on US 285, Poncha Springs

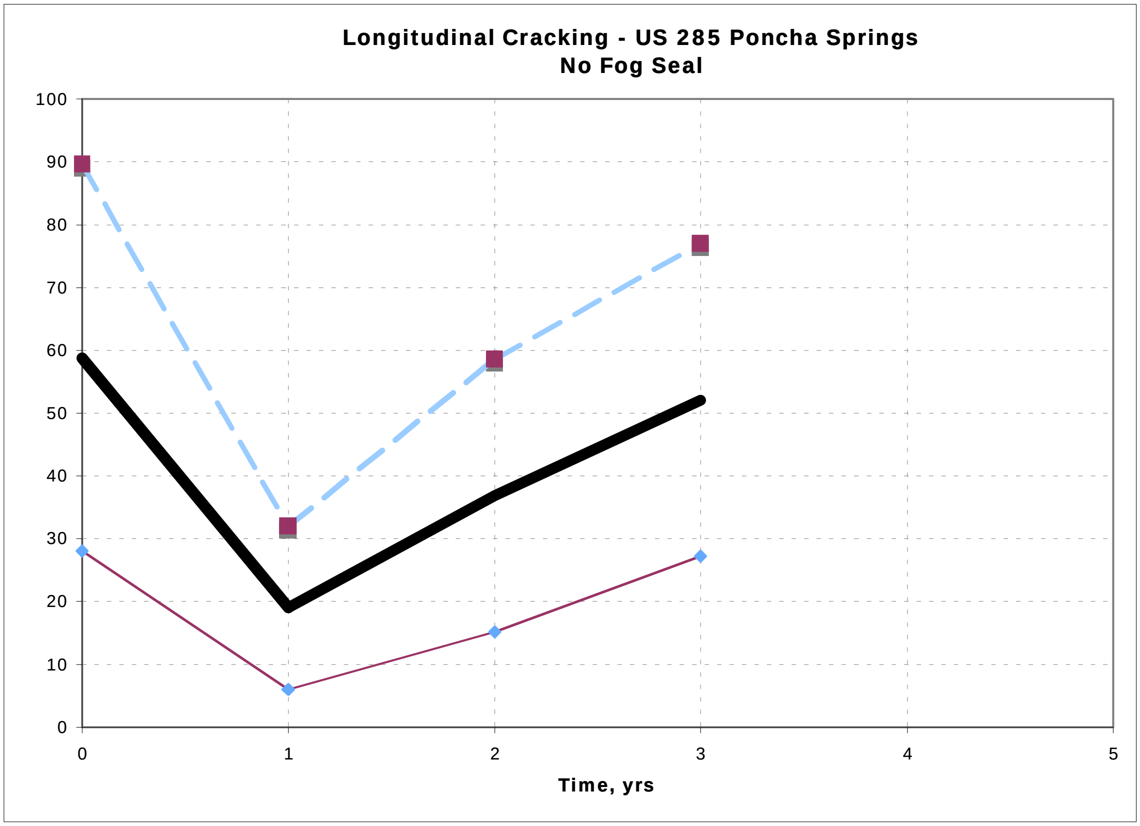


Figure 41. Longitudinal Cracking /500 ft Section for No Fog Sections on US 285, Poncha Springs

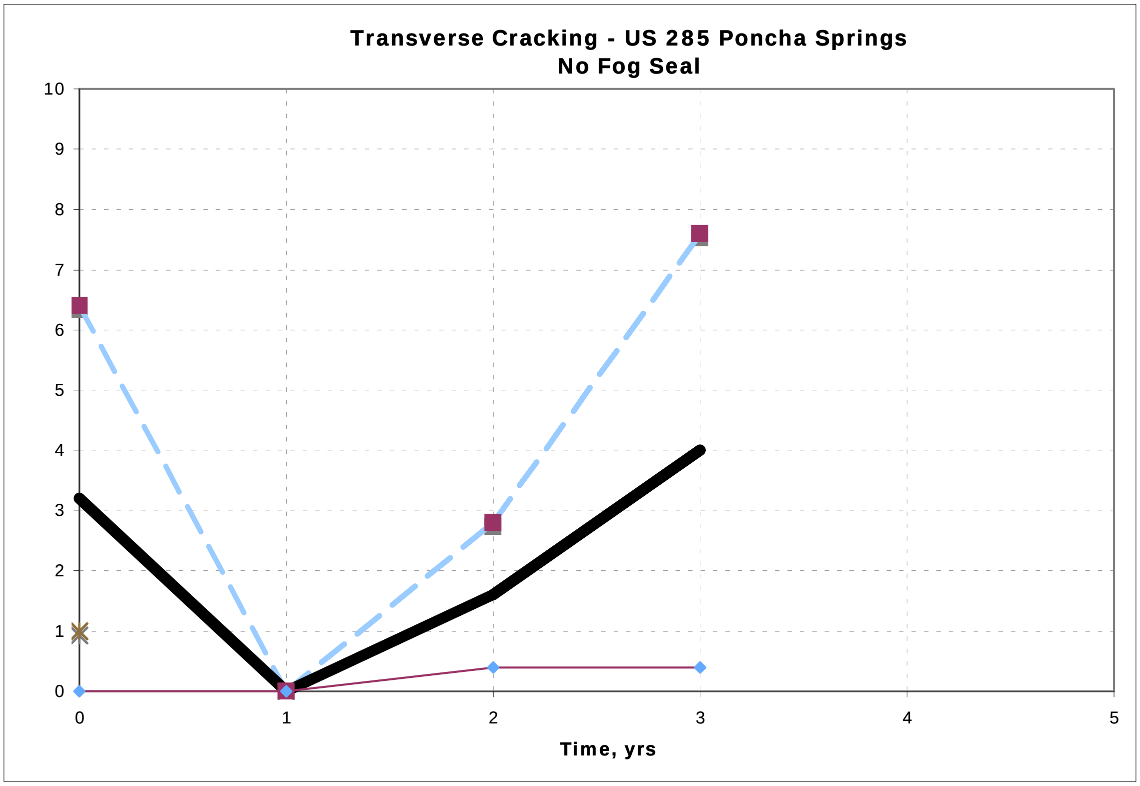


Figure 42. Transverse Cracking /500 ft Section for No Fog Sections on US 285, Poncha Springs

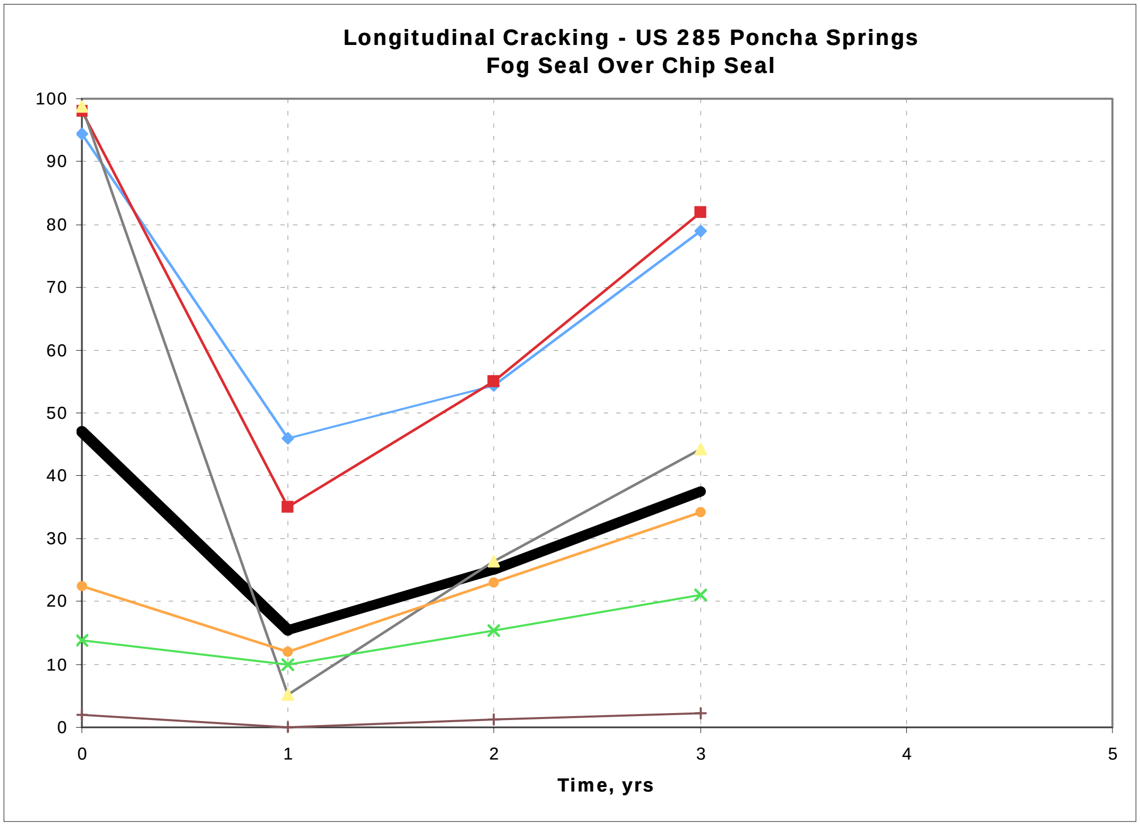


Figure 43. Longitudinal Cracking /500 ft Section for Fog Seal Sections on US 285, Poncha Springs

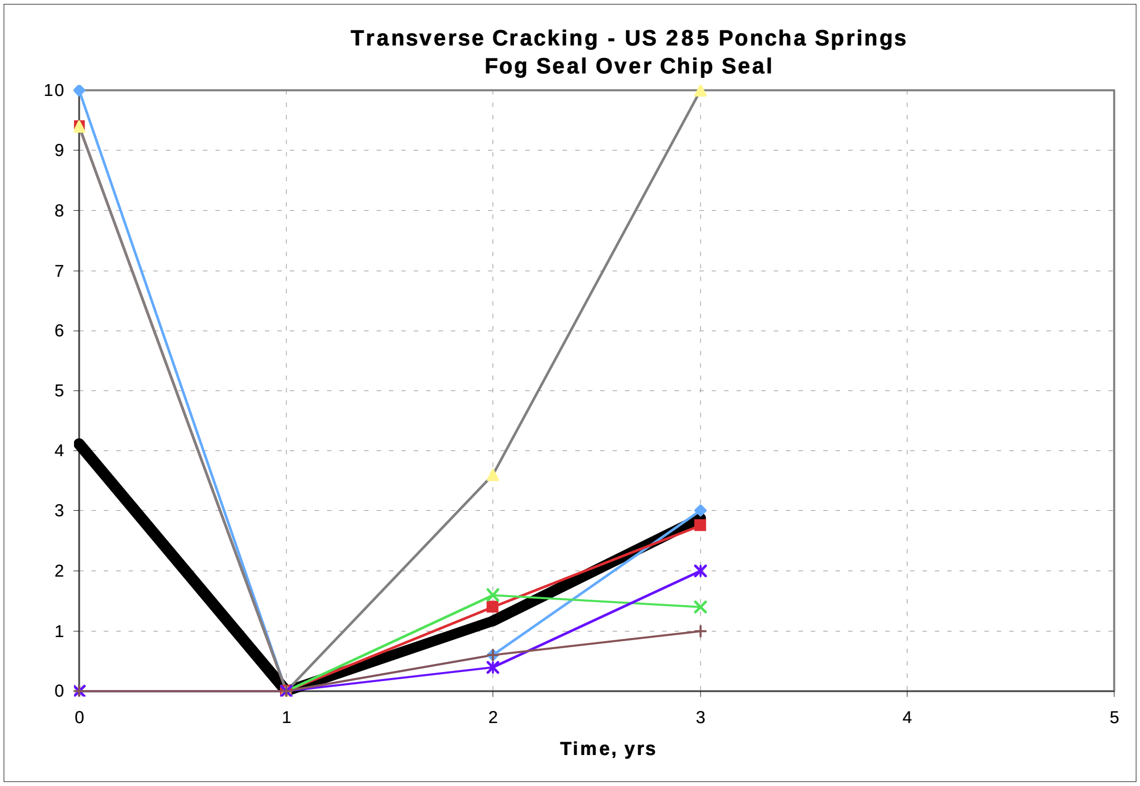


Figure 44. Transverse Cracking /500 ft Section for Fog Seal Sections on US 285, Poncha Springs

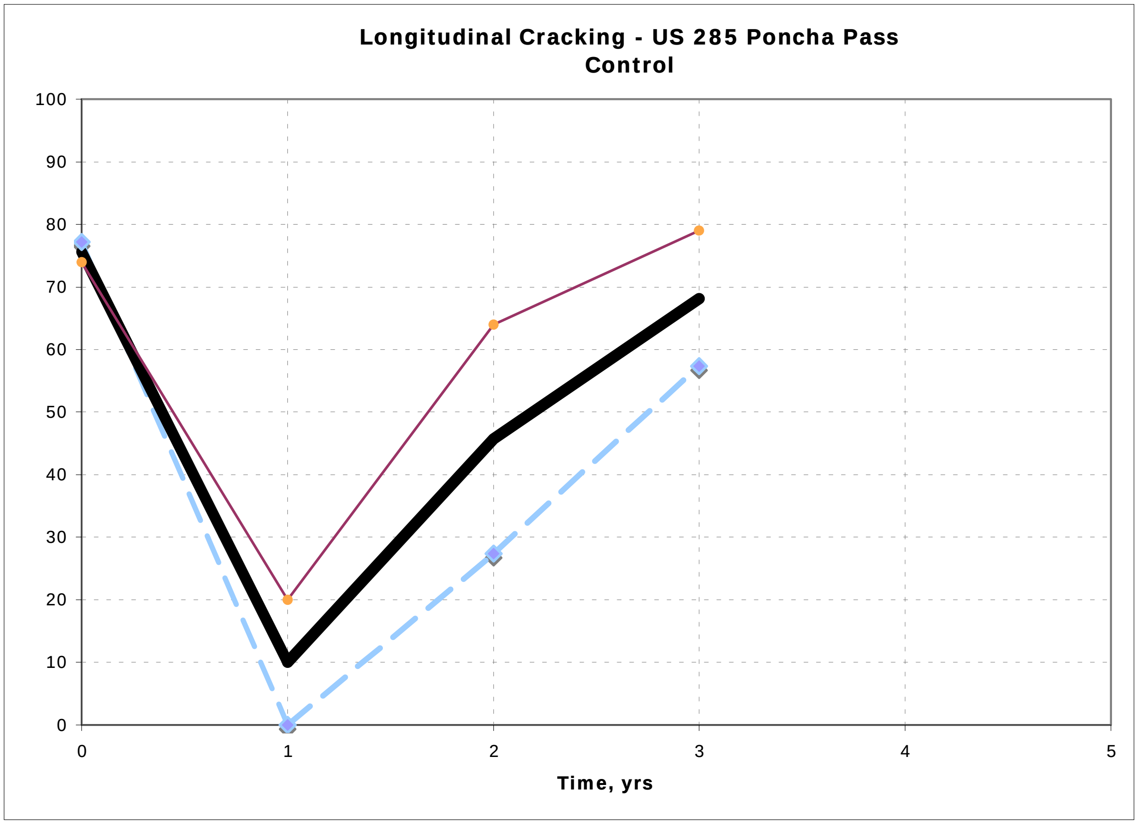


Figure 45. Longitudinal Cracking /500 ft Section for Control Sections on US 285, Poncha Pass

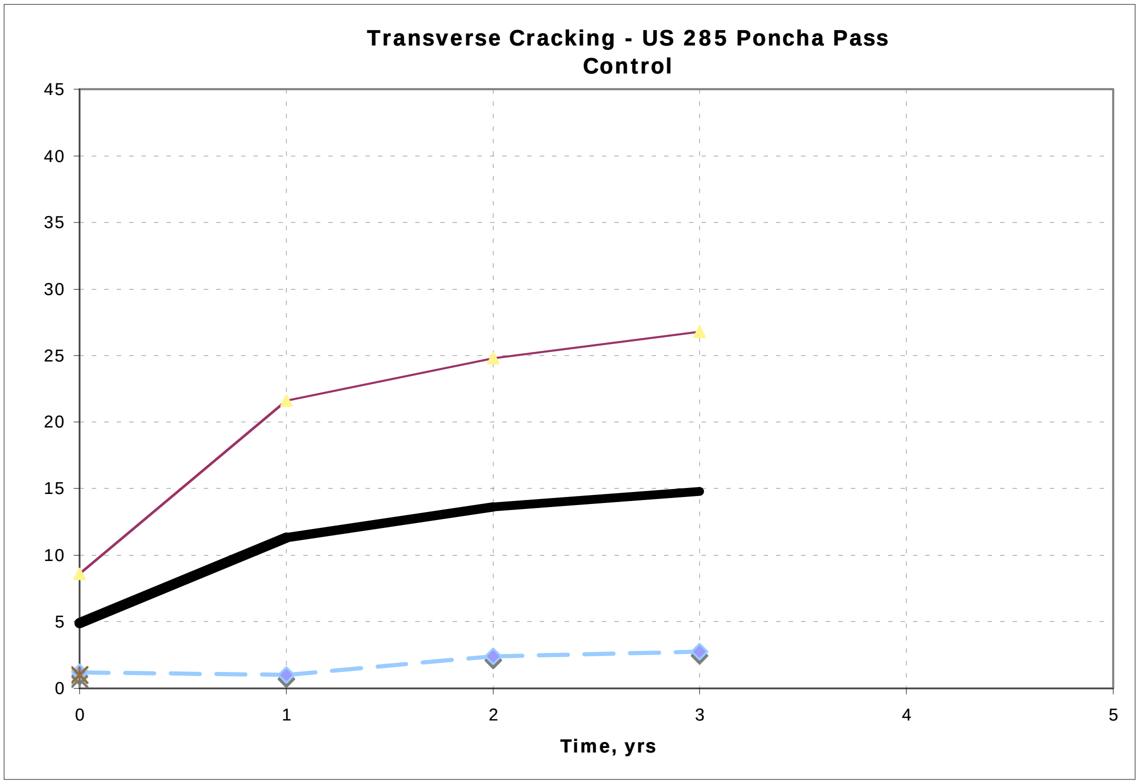


Figure 46. Transverse Cracking /500 ft Section for Control Sections on US 285, Poncha Pass

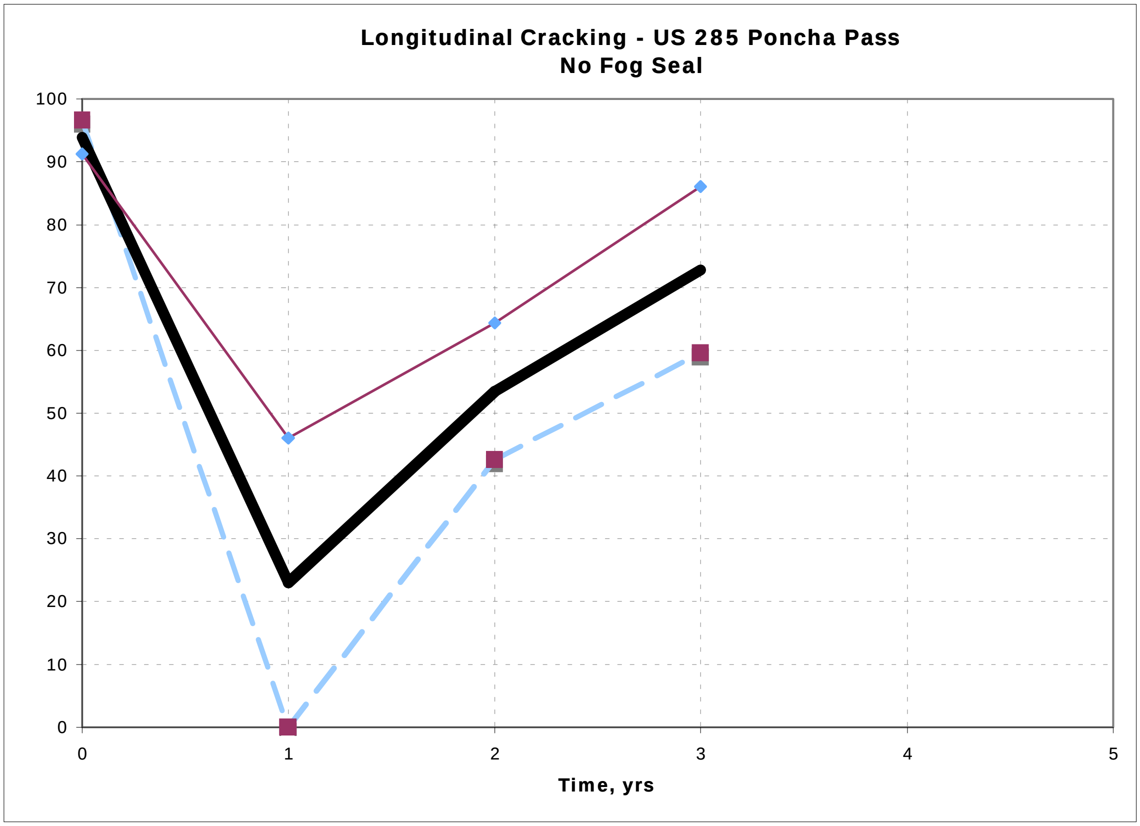


Figure 47. Longitudinal Cracking /500 ft Section for No Fog Seal Sections on US 285, Poncha Pass

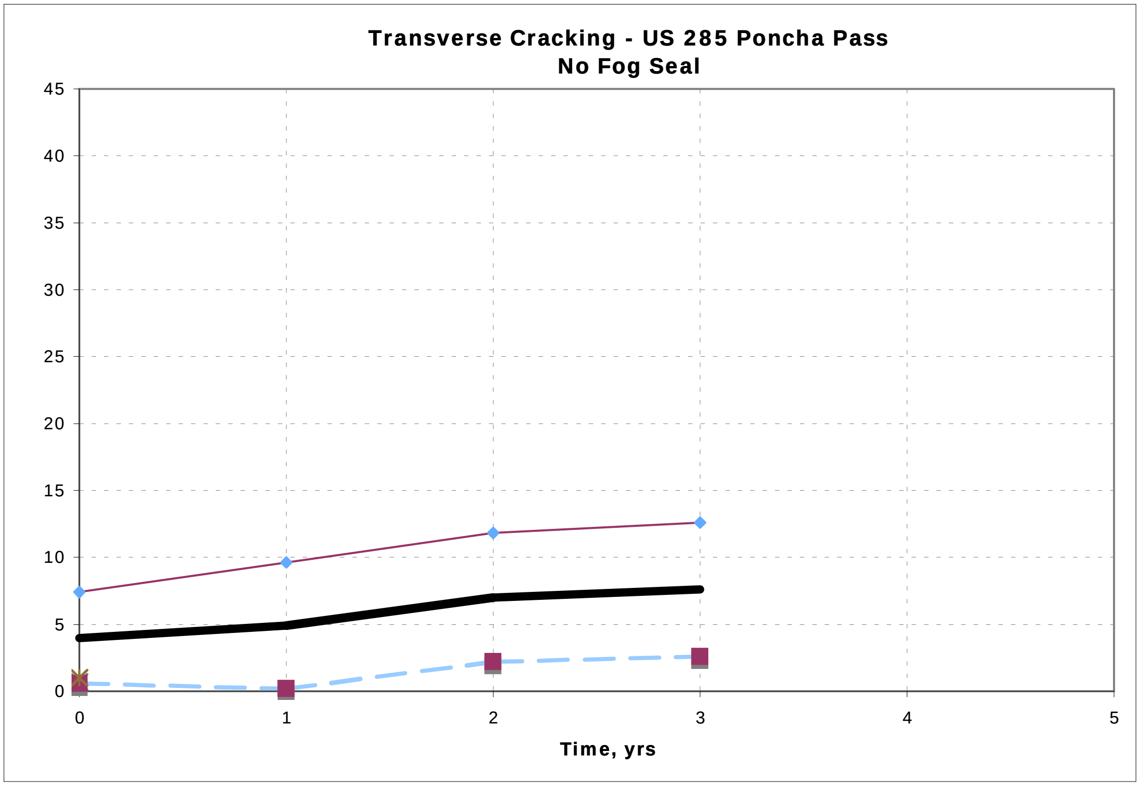


Figure 48. Transverse Cracking /500 ft Section for No Fog Seal Sections on US 285, Poncha Pass

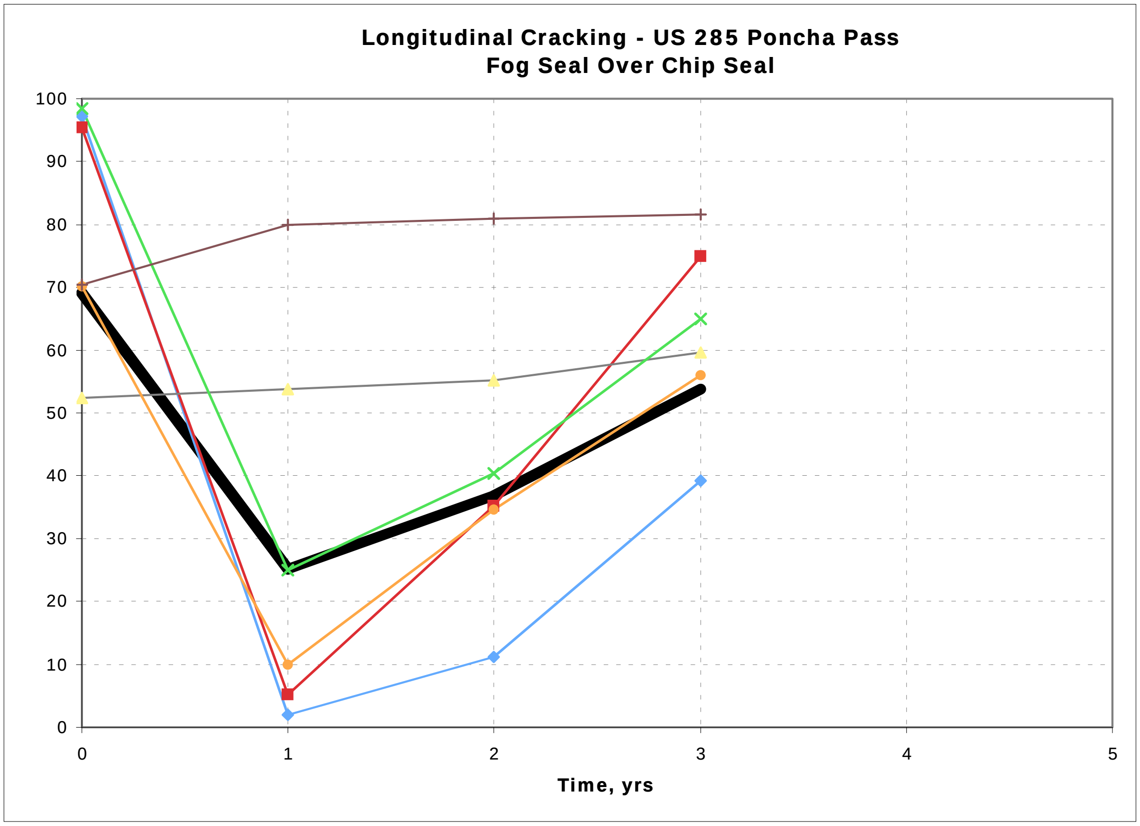


Figure 49. Longitudinal Cracking /500 ft Section for Fog Sealed Sections on US 285, Poncha Pass

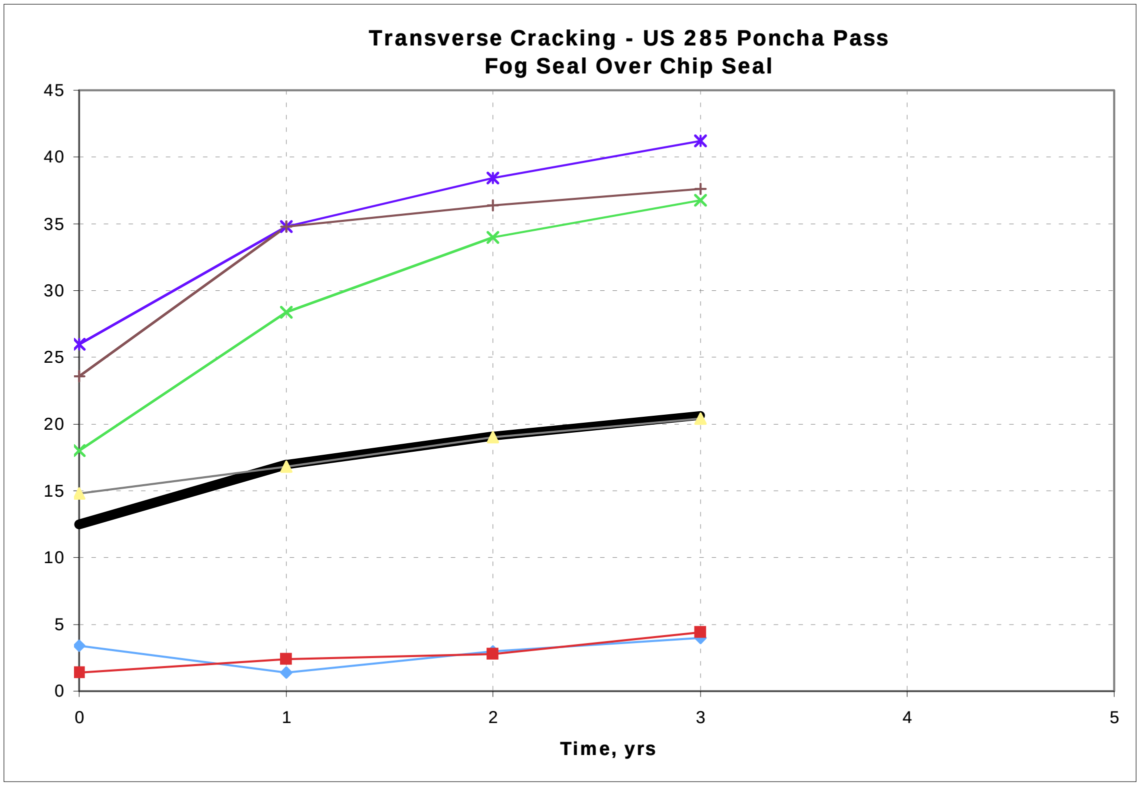


Figure 50. Transverse Cracking /500 ft Section for Fog Sealed Sections on US 285, Poncha Pass

4.3.4 US 24, Leadville

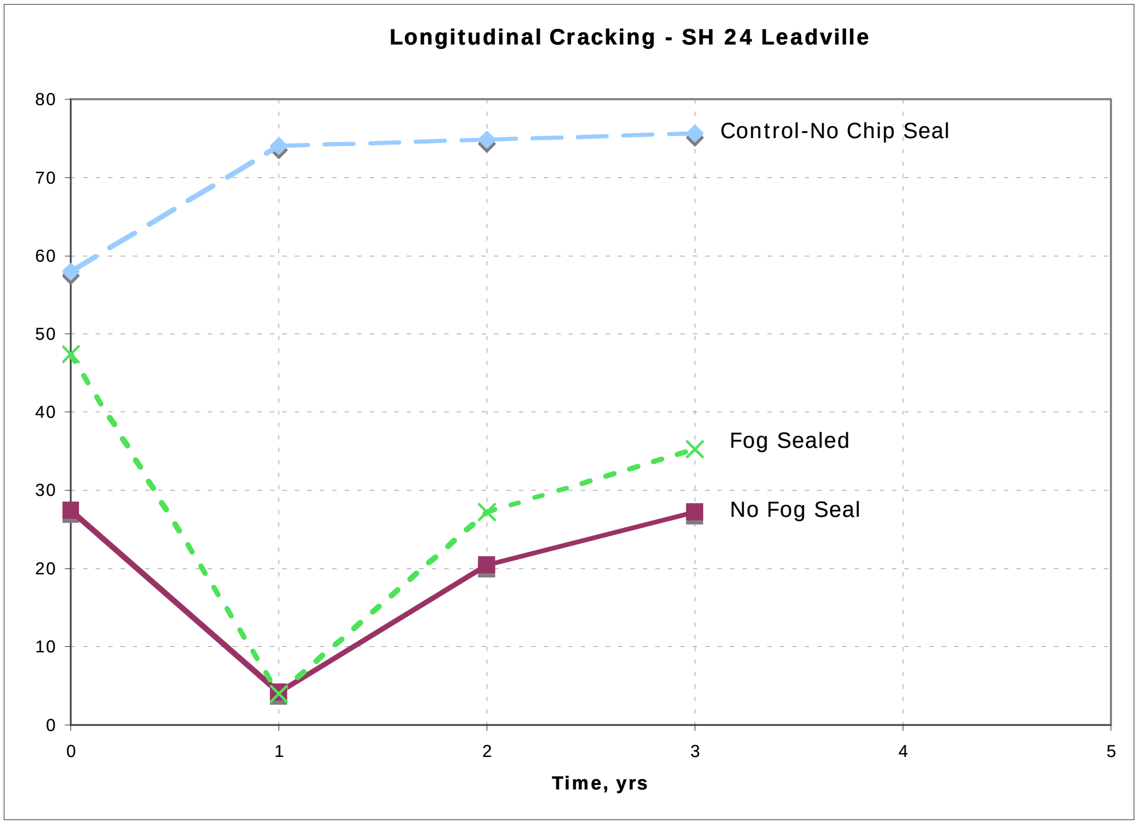


Figure 51. Longitudinal Cracking /500 ft Section for Test Sections on SH 24, Leadville

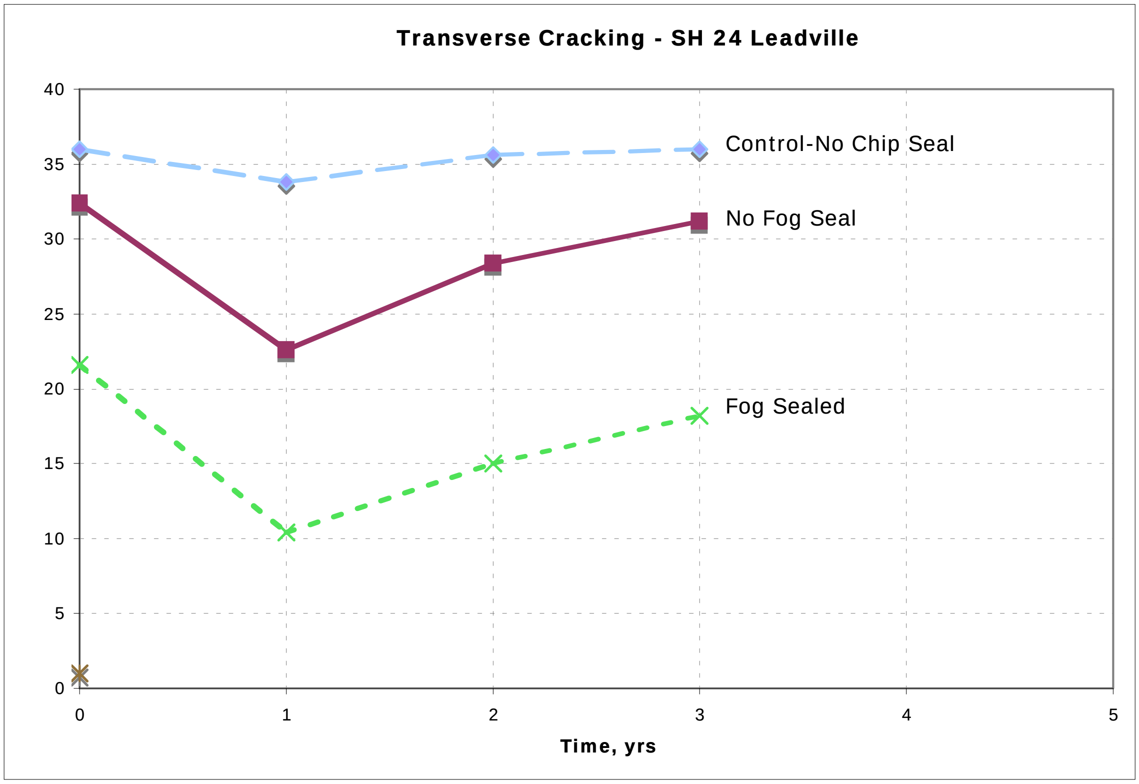


Figure 52. Transverse Cracking /500 ft Section for Test Sections on SH 24, Leadville

4.3.5 US50, Swink

There are eight evaluation sections on US 50 near Swink, two each in the eastbound and westbound driving lanes and two each in the east and westbound passing lanes. Results of performance evaluations are shown in Figures 53 through 60 for each section. Average condition is shown as the solid bold line on each figure.

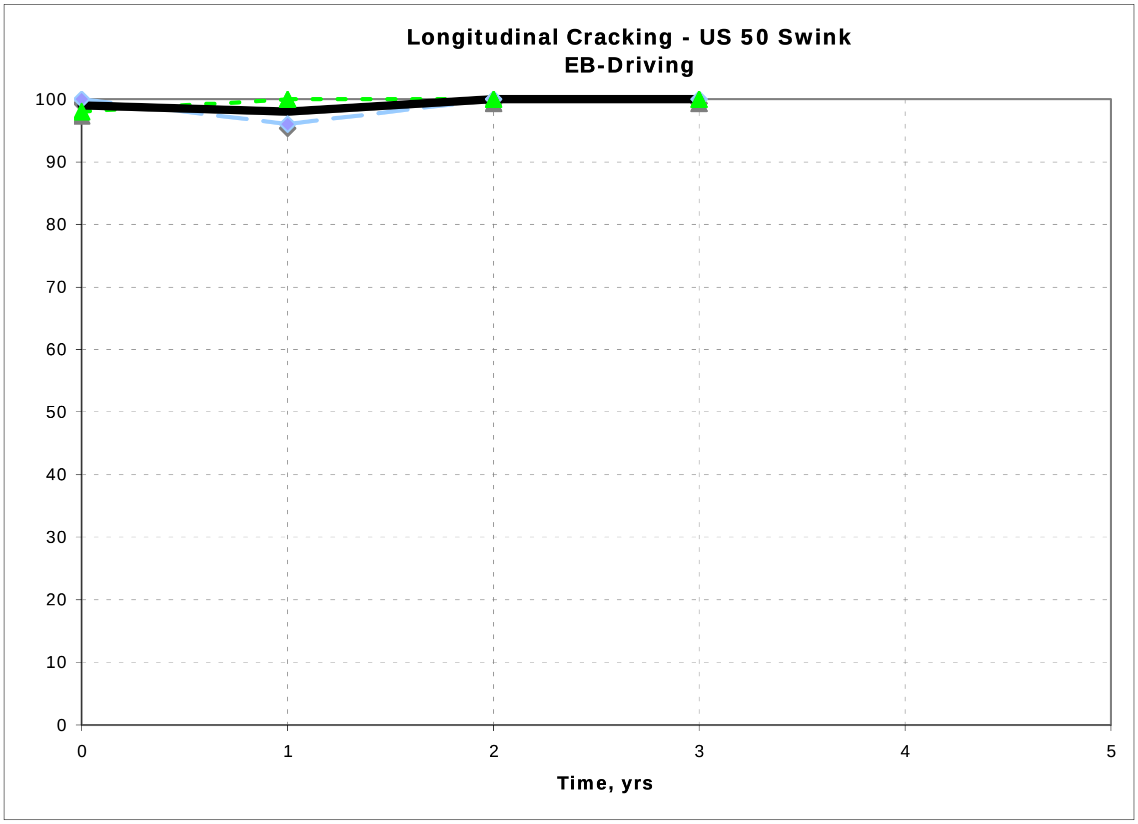


Figure 53. Longitudinal Cracking /500 ft Section for EB Driving Lane-US 50, Swink

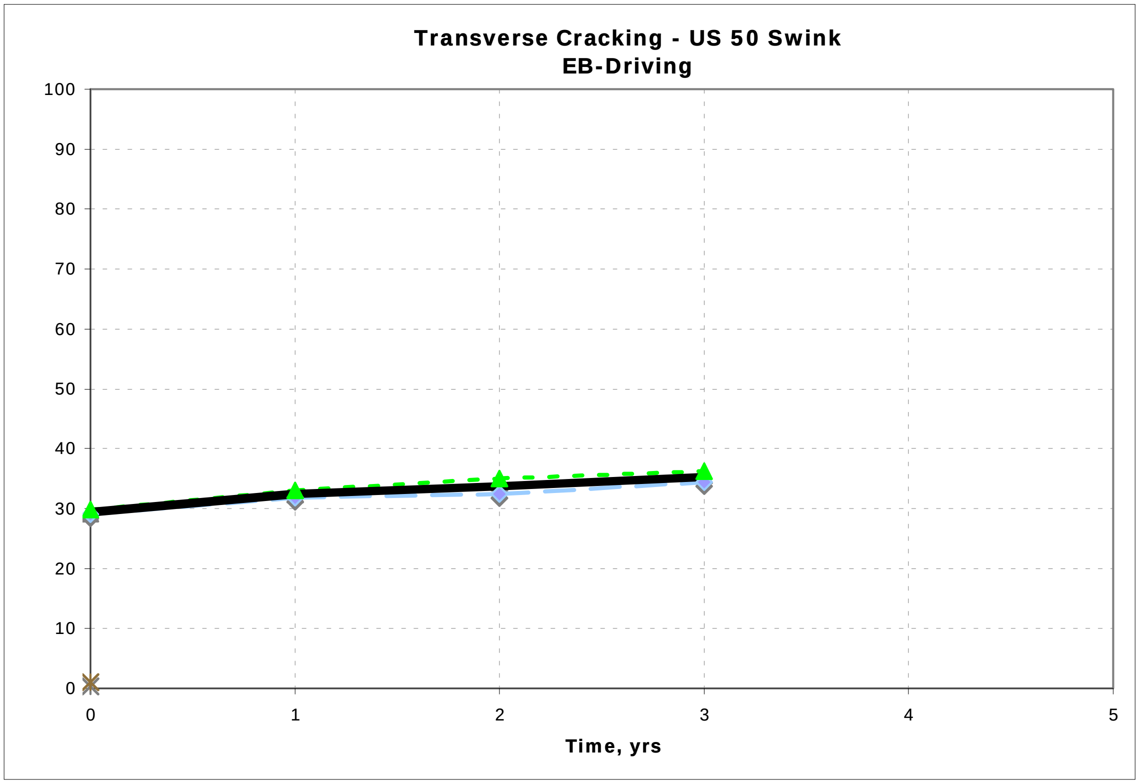


Figure 54. Transverse Cracking /500 ft Section for EB Driving Lane-US 50, Swink

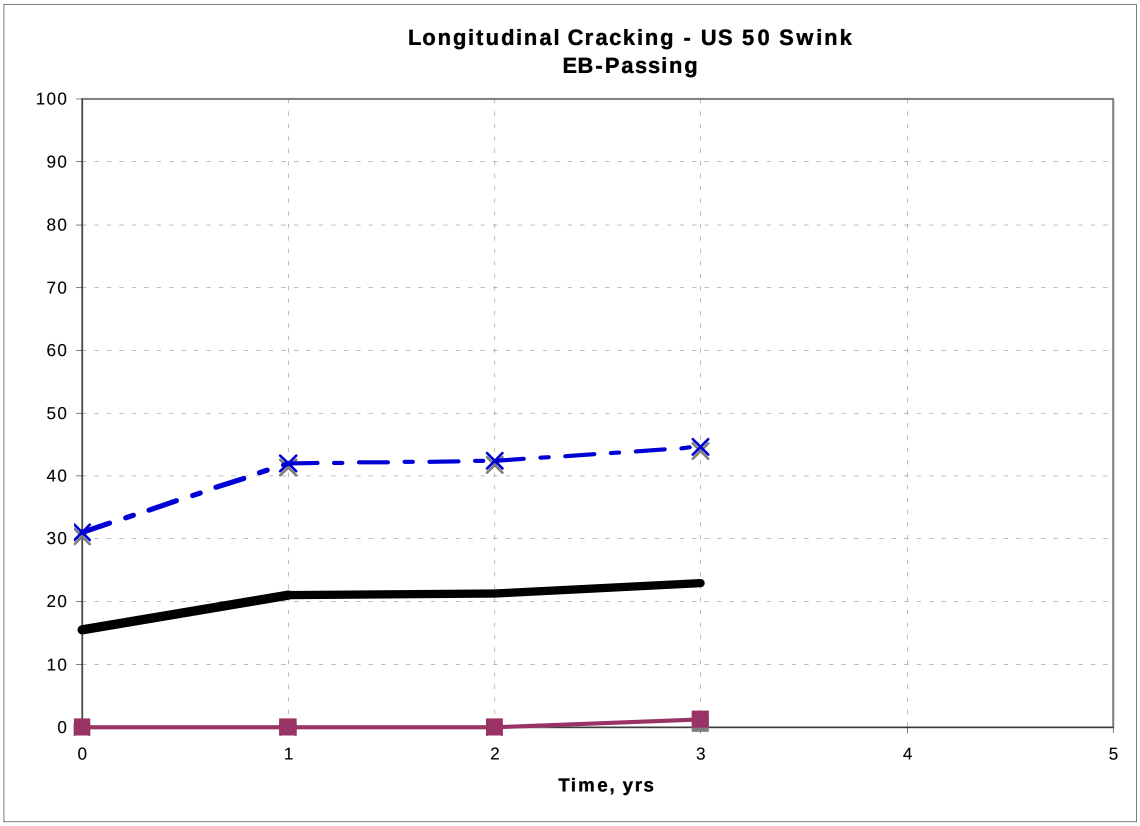


Figure 55. Longitudinal Cracking /500 ft Section for EB Passing Lane-US 50, Swink

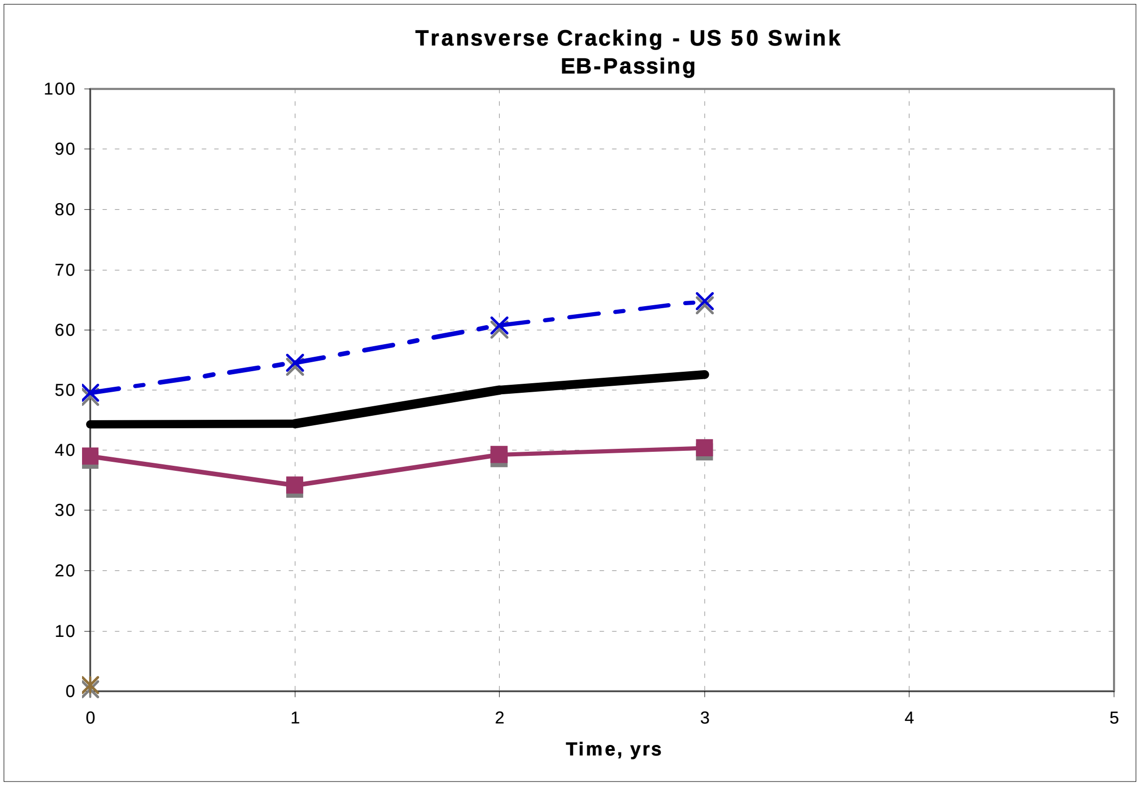


Figure 56. Transverse Cracking /500 ft Section for EB Passing Lane-US 50, Swink

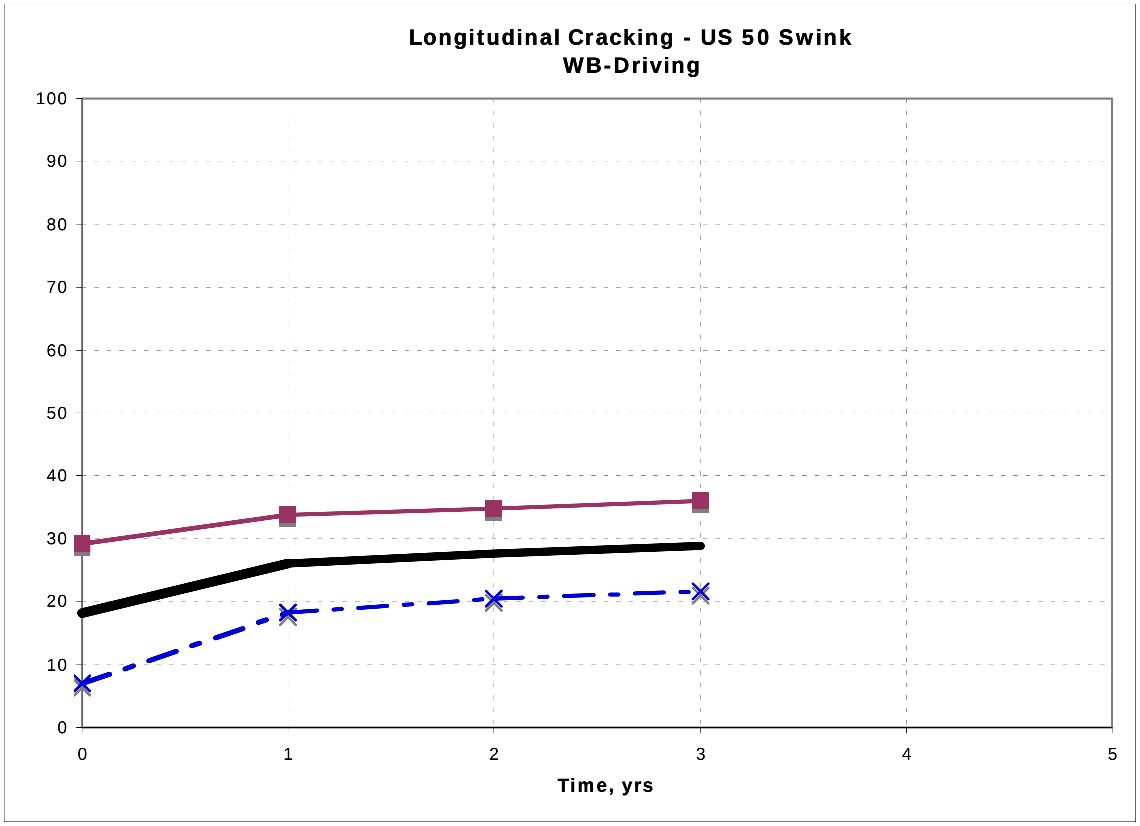


Figure 57. Longitudinal Cracking /500 ft Section for WB Driving Lane-US 50, Swink

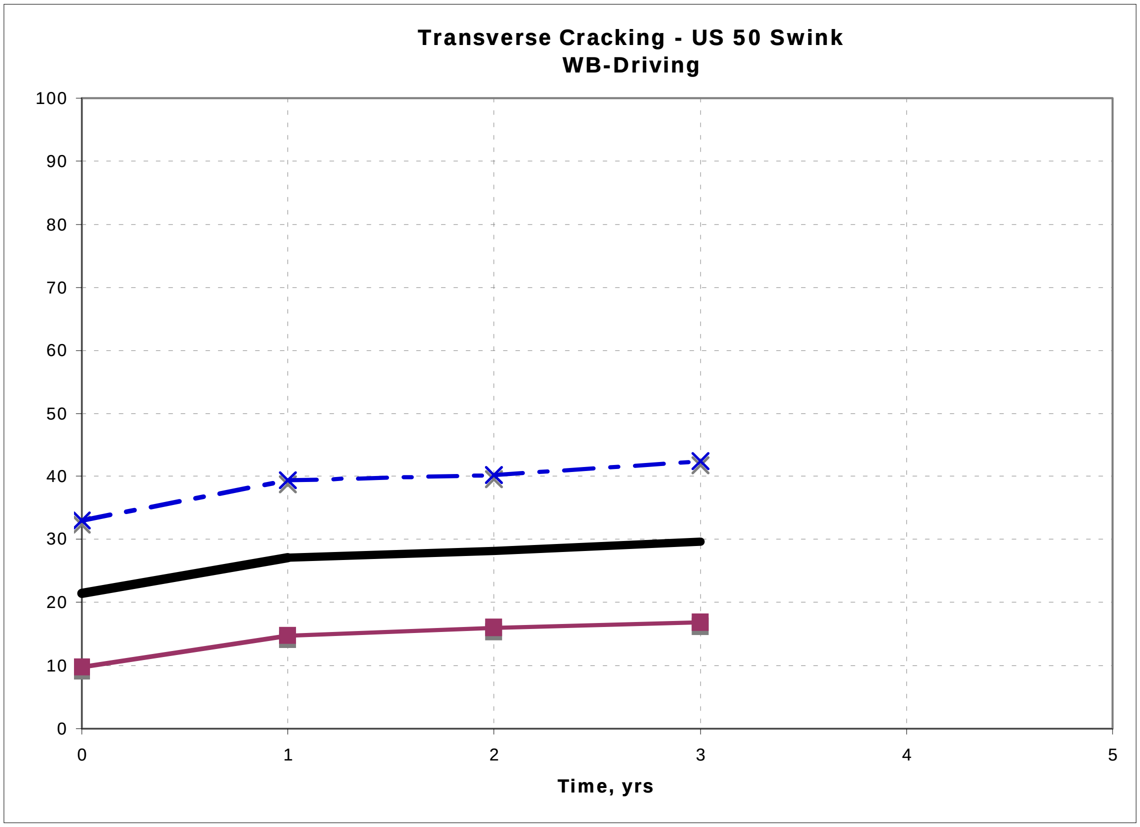


Figure 58. Transverse Cracking /500 ft Section for WB Driving Lane-US 50, Swink

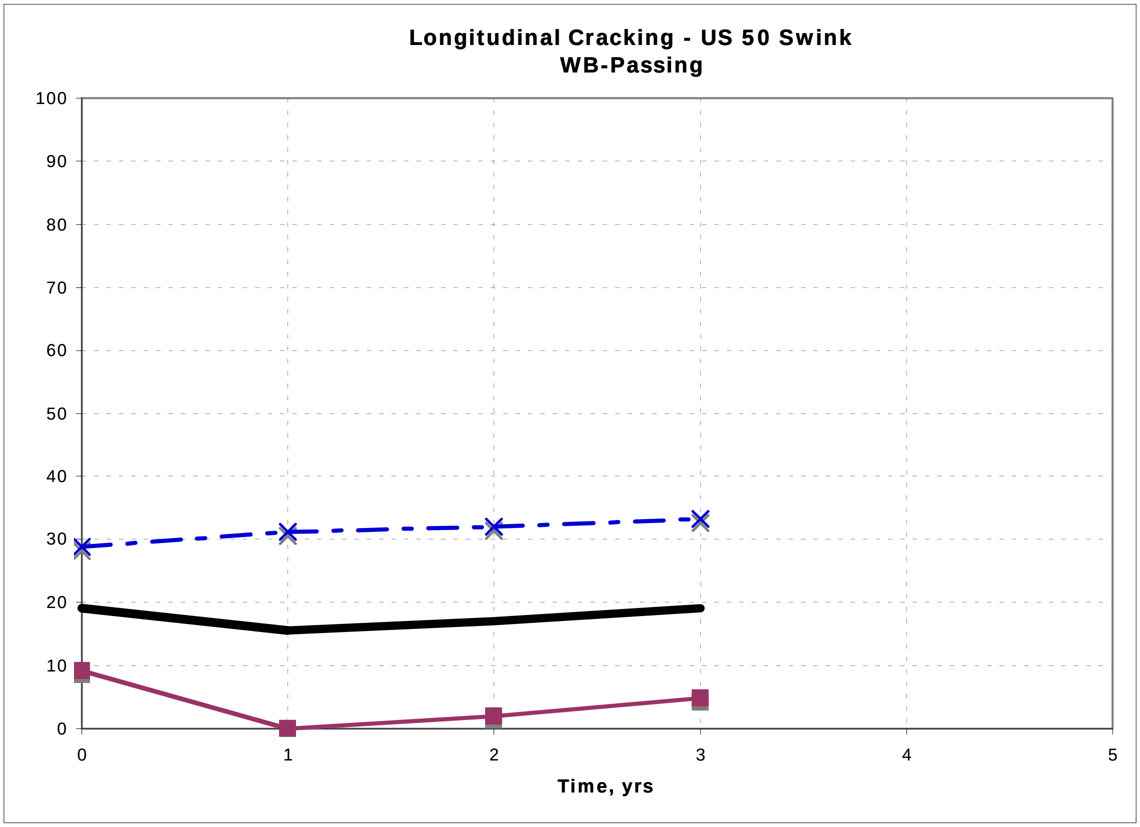


Figure 59. Longitudinal Cracking /500 ft Section for WB Passing Lane-US 50, Swink

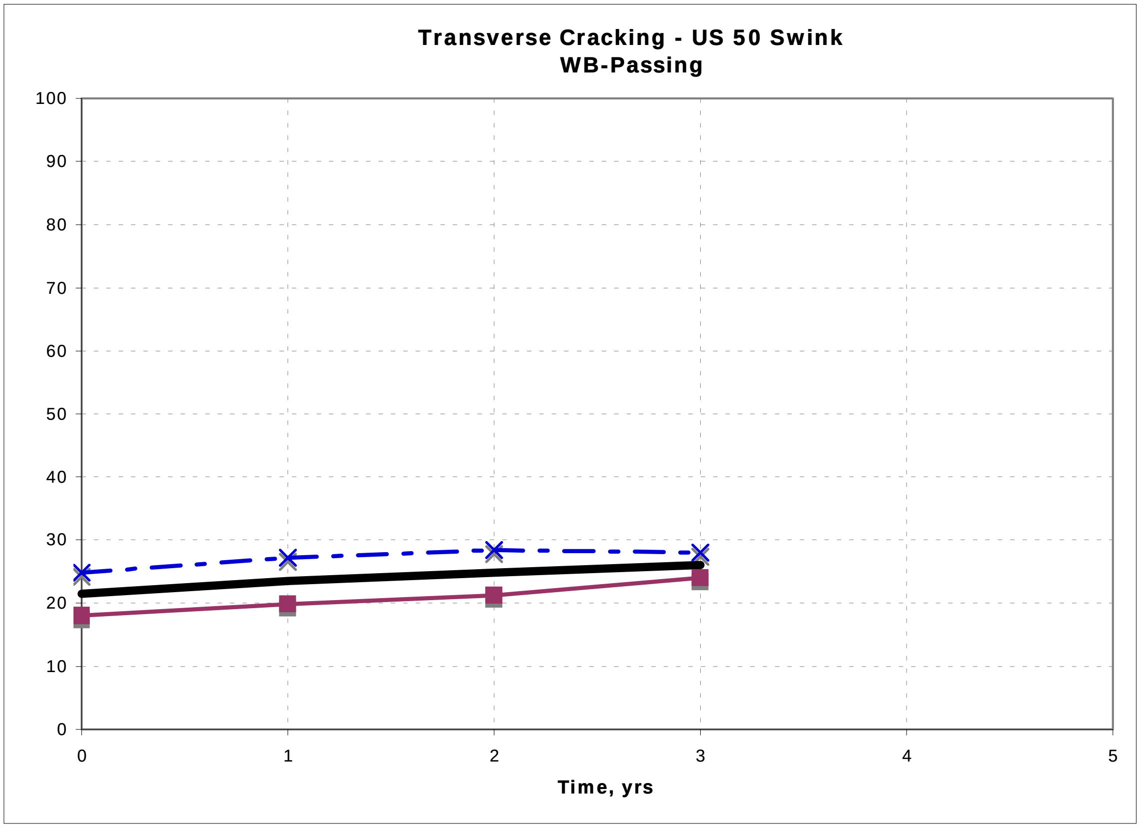


Figure 60. Transverse Cracking /500 ft Section for WB Passing Lane-US 50, Swink

4.4 Thin Hot Mix Overlays

4.4.1 US 6, Golden

Two evaluation sections were established on US 6, Golden. The results of two years of performance data on both sections and the average are shown in Figures 61 through 63. Two years of performance data was collected before the site was widened and overlayed.

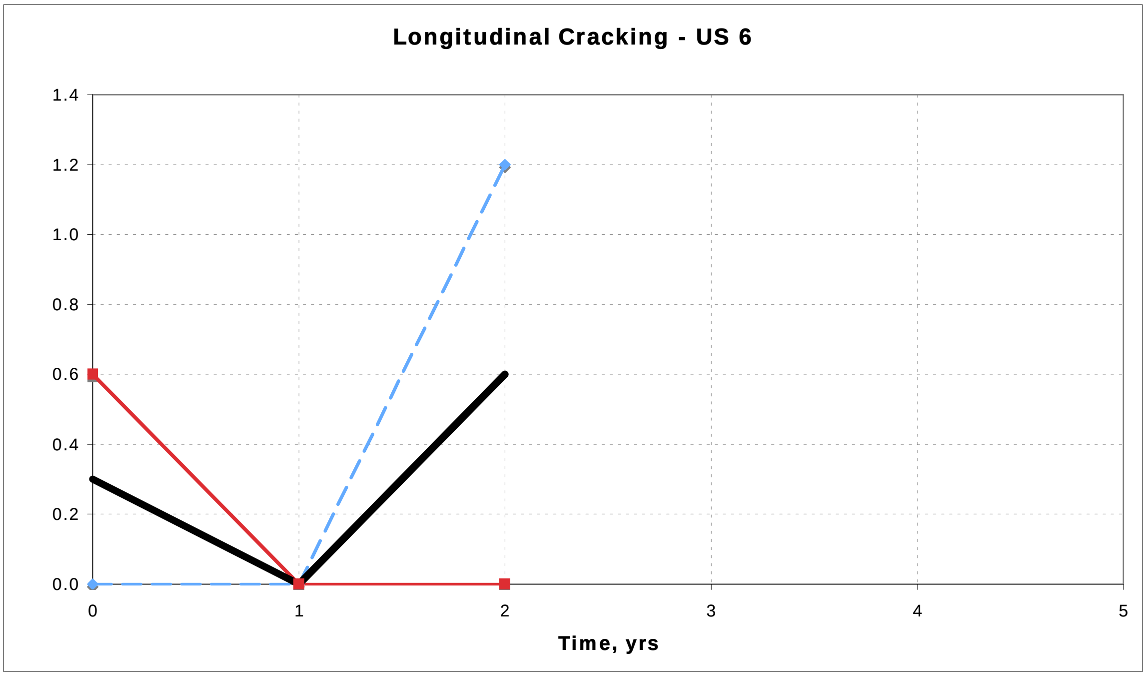


Figure 61. Longitudinal Cracking/500 ft Section for US 6, Golden

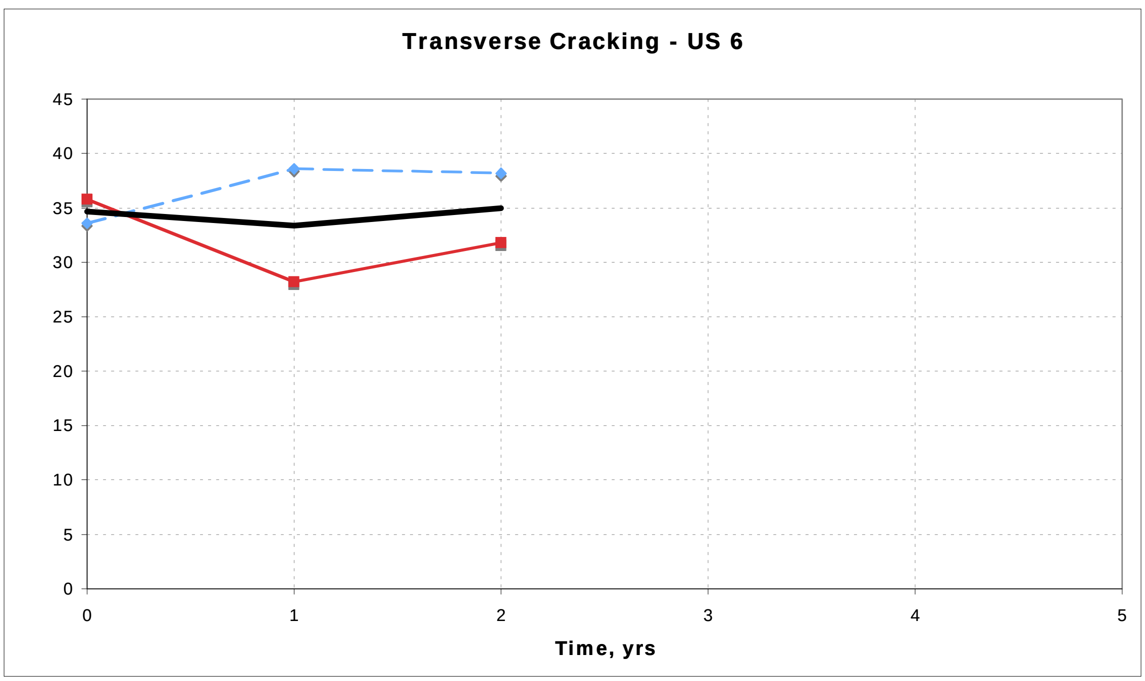


Figure 62. Transverse Cracking /500 ft Section for US 6, Golden

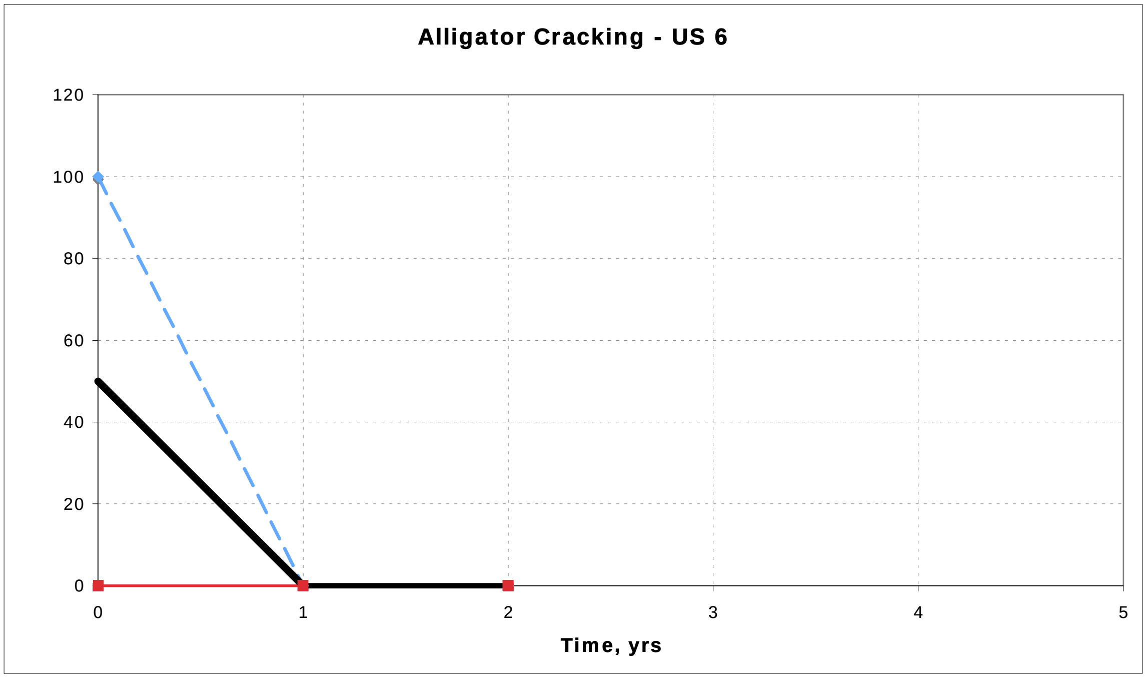


Figure 63. Alligator Cracking/500 ft Section for US 6, Golden

4.4.2 SH121, Littleton

Two evaluation sections were established on SH 121, Littleton. The results of one year of performance data on both sections and the average are shown in Figures 64 and 65. One year of performance data was collected before another overlay was applied to the pavement.

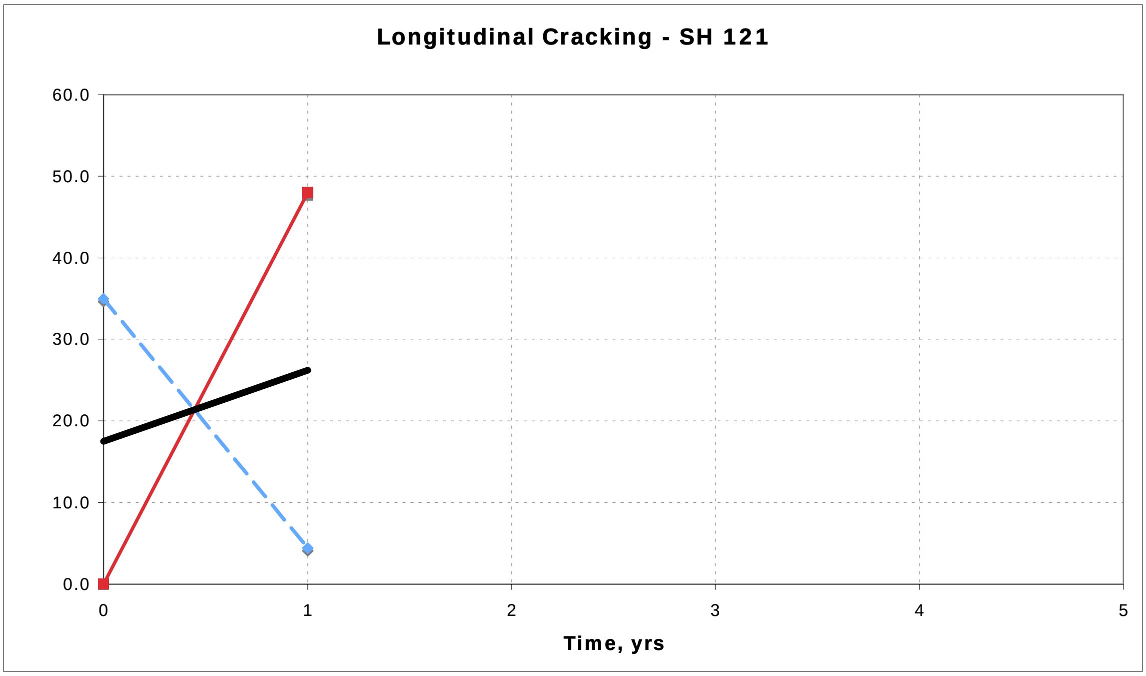


Figure 64. Longitudinal Cracking/500 ft Section for SH 121 Littleton

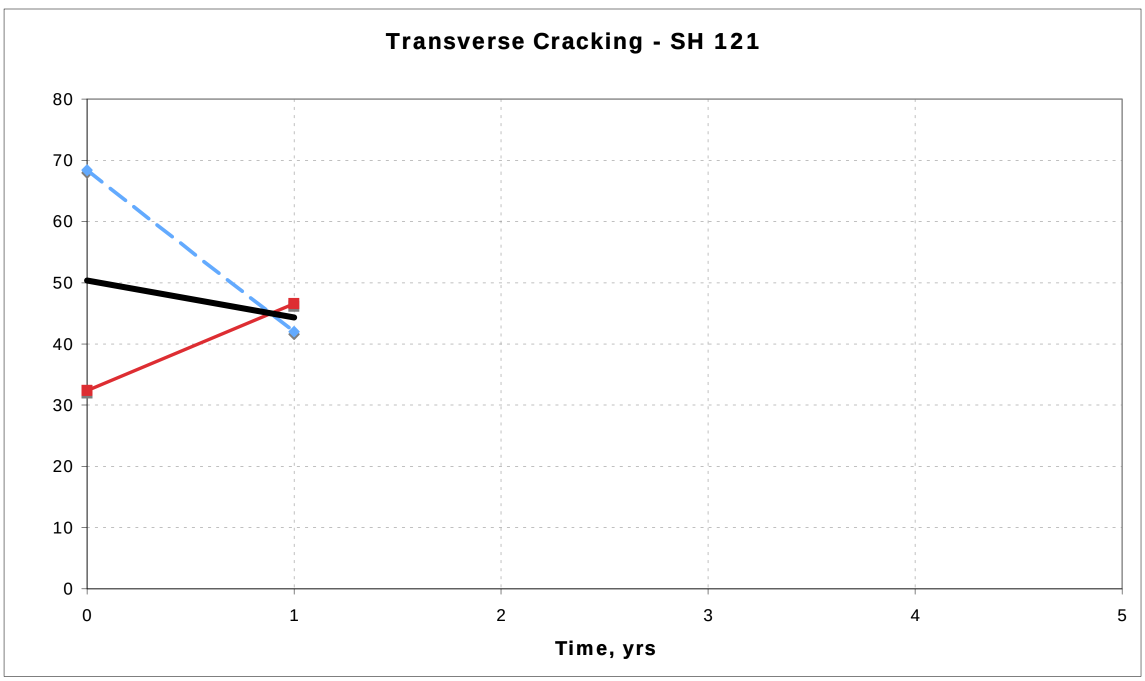


Figure 65. Transverse Cracking/500 ft Section for SH 121, Littleton

4.4.3 US550, Durango

Thirty evaluation sections were established at this site. Treatments evaluated include hot in-place recycling (HIPR), overlay thickness, and lane. The twelve combinations of these variables placed are shown in Table 10. Performance of the test sections will be presented with respect to these twelve combinations of variables so that 2 inch overlays over HIPR on the shoulders are not compared on the same graph as the 1.5 inch overlays over no HIPR on the passing lane. Figures 66 through 100 represent the performance of the test sections on the shoulders, driving lanes and passing lanes, respectively.

Table 10. Combinations of Variables Evaluated on US550 Durango

	Section	HIPR	Thickness, in
Shoulders	1	No	2
	5	No	2
	21	No	2
	25	No	2
	26	No	2
	30	No	2
	16	No	1.5
	20	No	1.5
	6	Yes	2
	10	Yes	2
	11	Yes	1.5
	15	Yes	1.5
Driving Lanes	22	No	2
	24	No	2
	17	No	1.5
	19	No	1.5
	2	Yes	2
	4	Yes	2
	7	Yes	2
	9	Yes	2
	27	Yes	2
	29	Yes	2
	12	Yes	1.5
	14	Yes	1.5
Passing Lane	23	No	2
	18	No	1.5
	3	Yes	2
	8	Yes	2
	28	Yes	2
	13	Yes	1.5

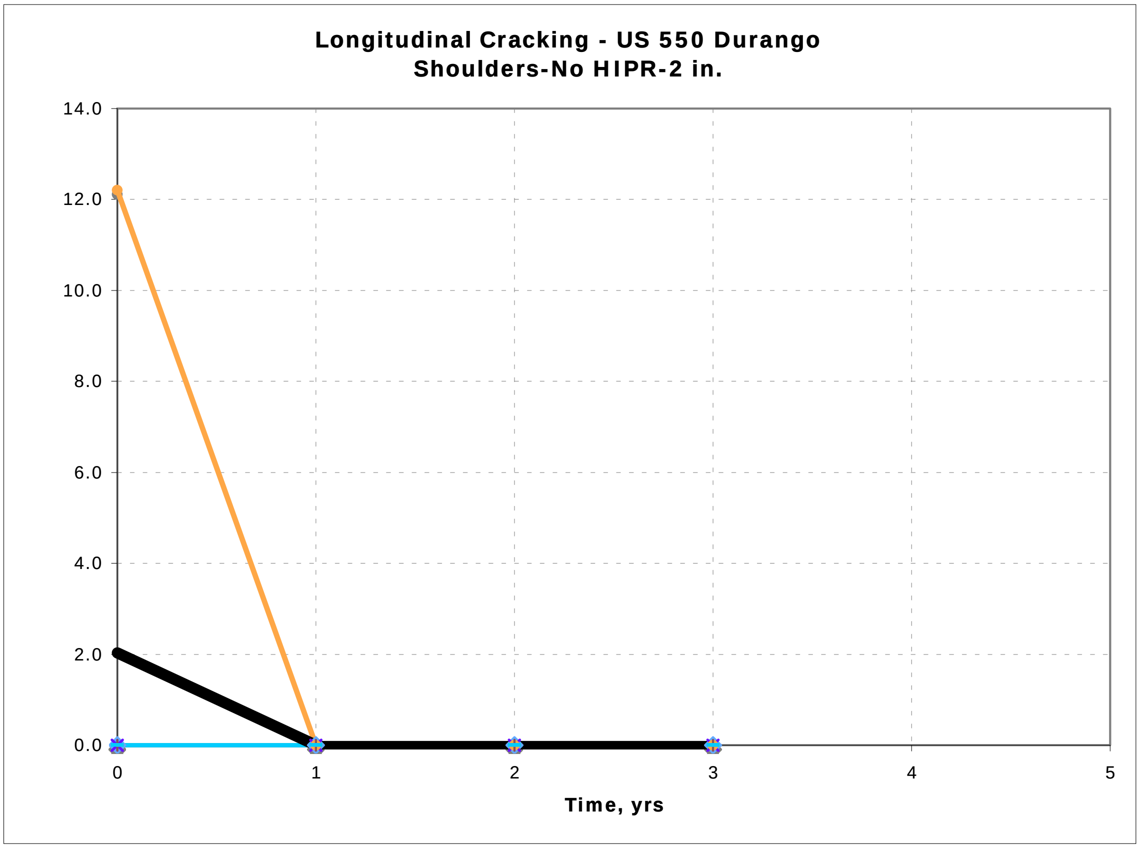


Figure 66. Longitudinal Cracking/500 ft Section for US 550, Durango – Shoulders, No HIPR, 2 in.

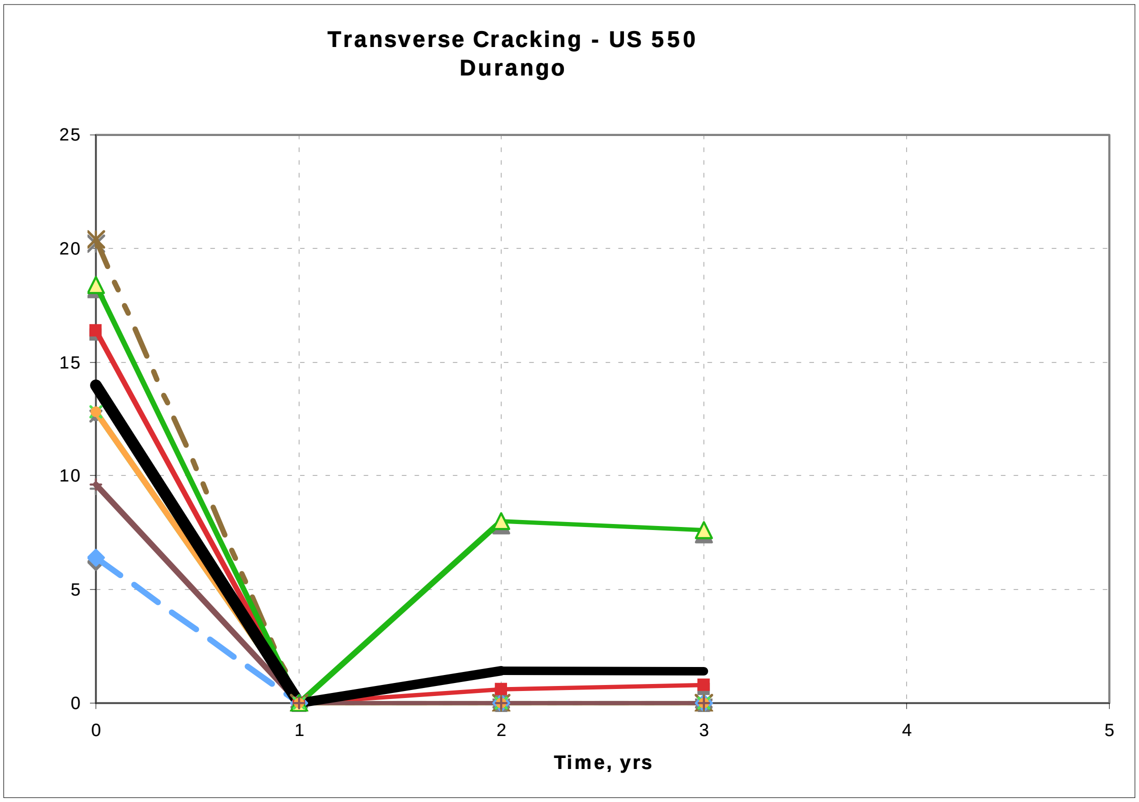


Figure 67. Transverse Cracking/500 ft Section for US 550, Durango – Shoulders, No HIPR, 2 in.

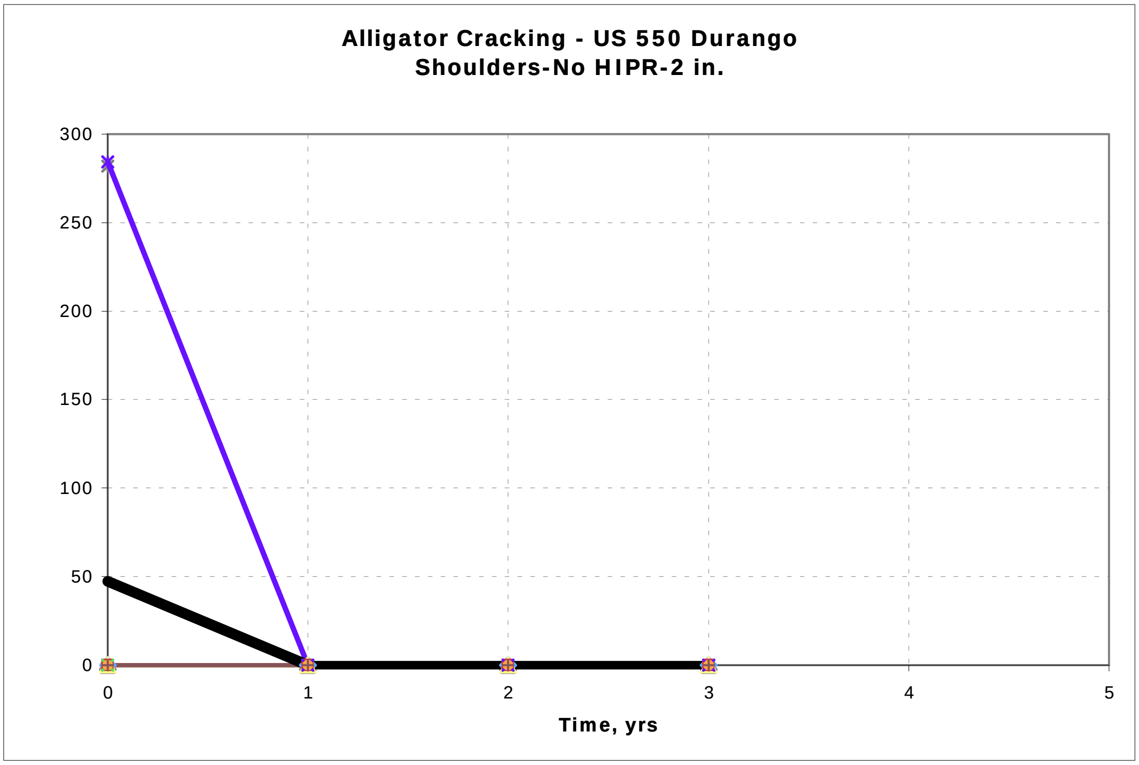


Figure 68. Alligator Cracking/500 ft Section for US 550, Durango – Shoulders, No HIPR, 2 in.

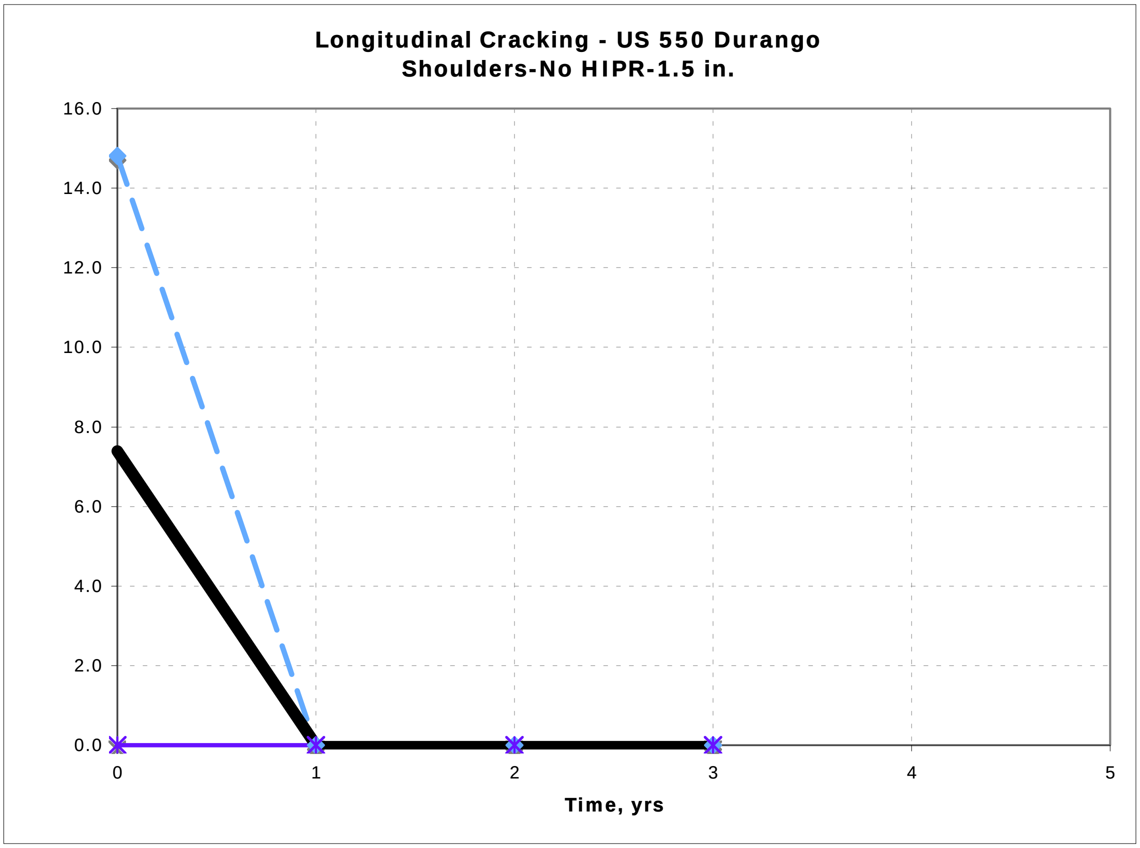


Figure 69. Longitudinal Cracking/500 ft Section for US 550, Durango – Shoulders, No HIPR, 1.5 in.

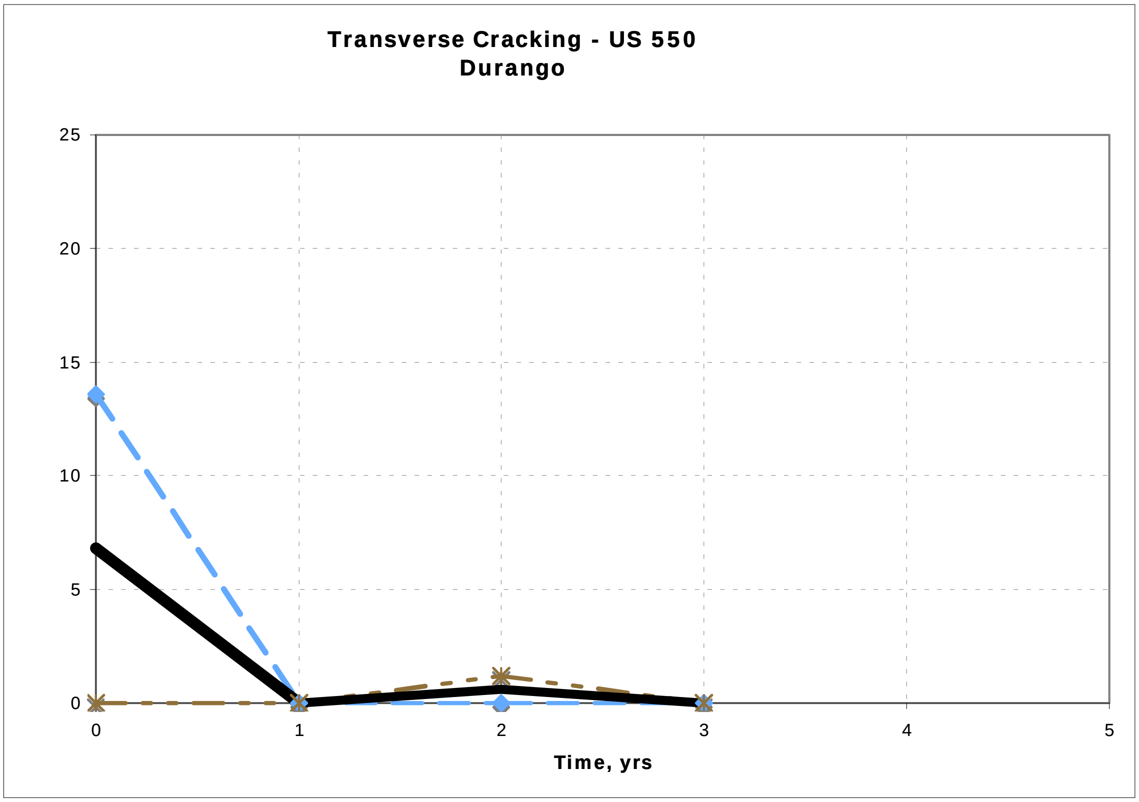


Figure 70. Transverse Cracking/500 ft Section for US 550, Durango – Shoulders, No HIPR, 1.5 in.

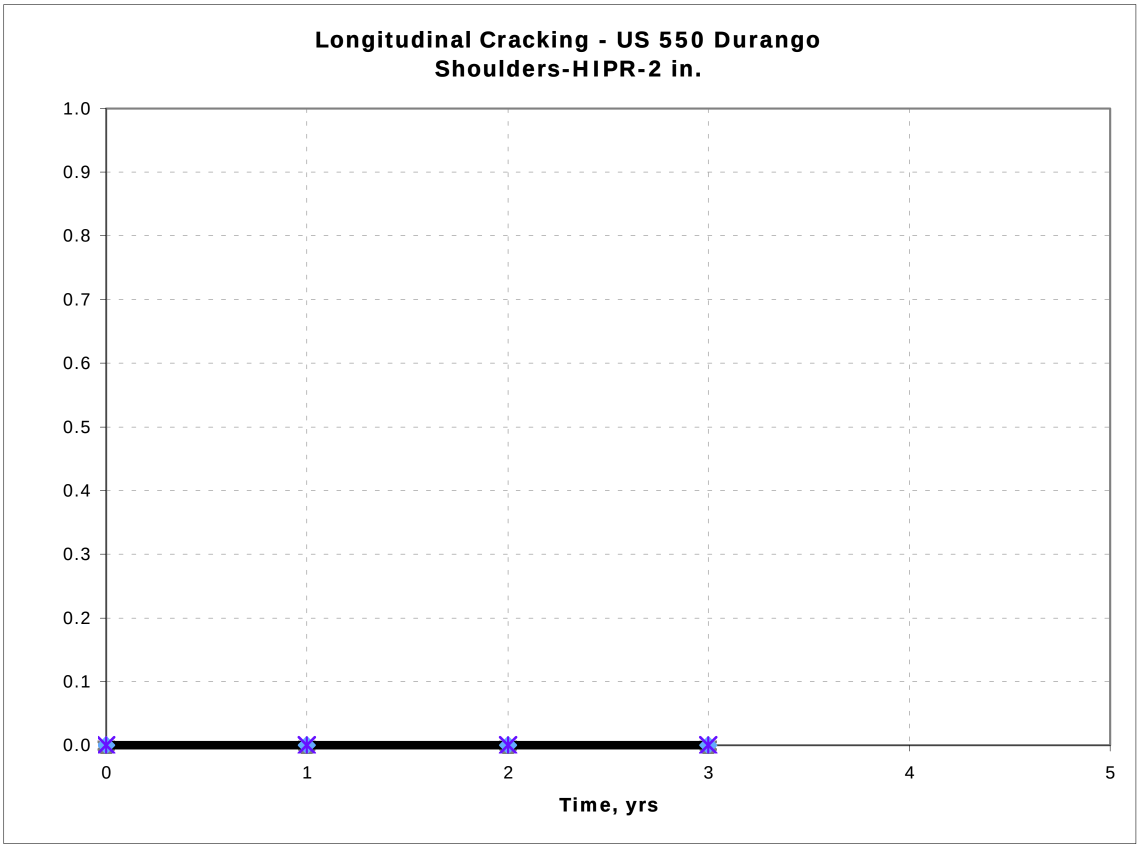


Figure 71. Longitudinal Cracking/500 ft Section for US 550, Durango – Shoulders, HIPR, 2 in.

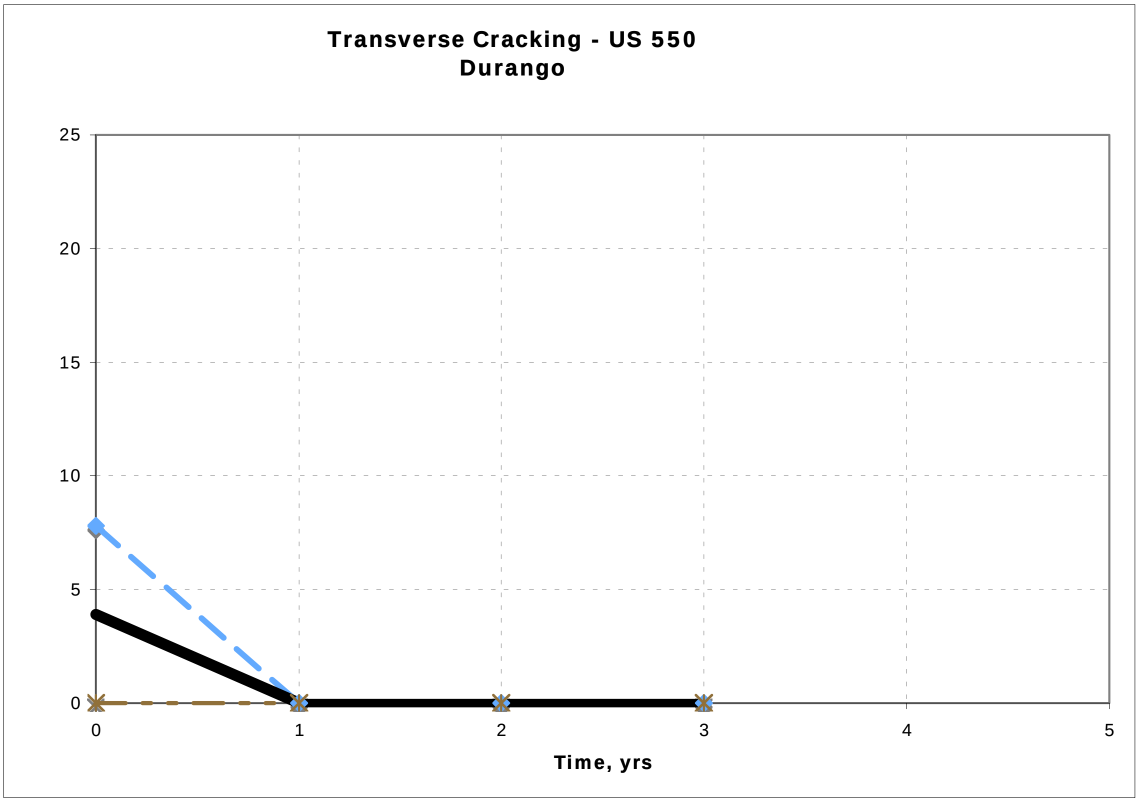


Figure 72. Transverse Cracking/500 ft Section for US 550, Durango – Shoulders, HIPR, 2 in.

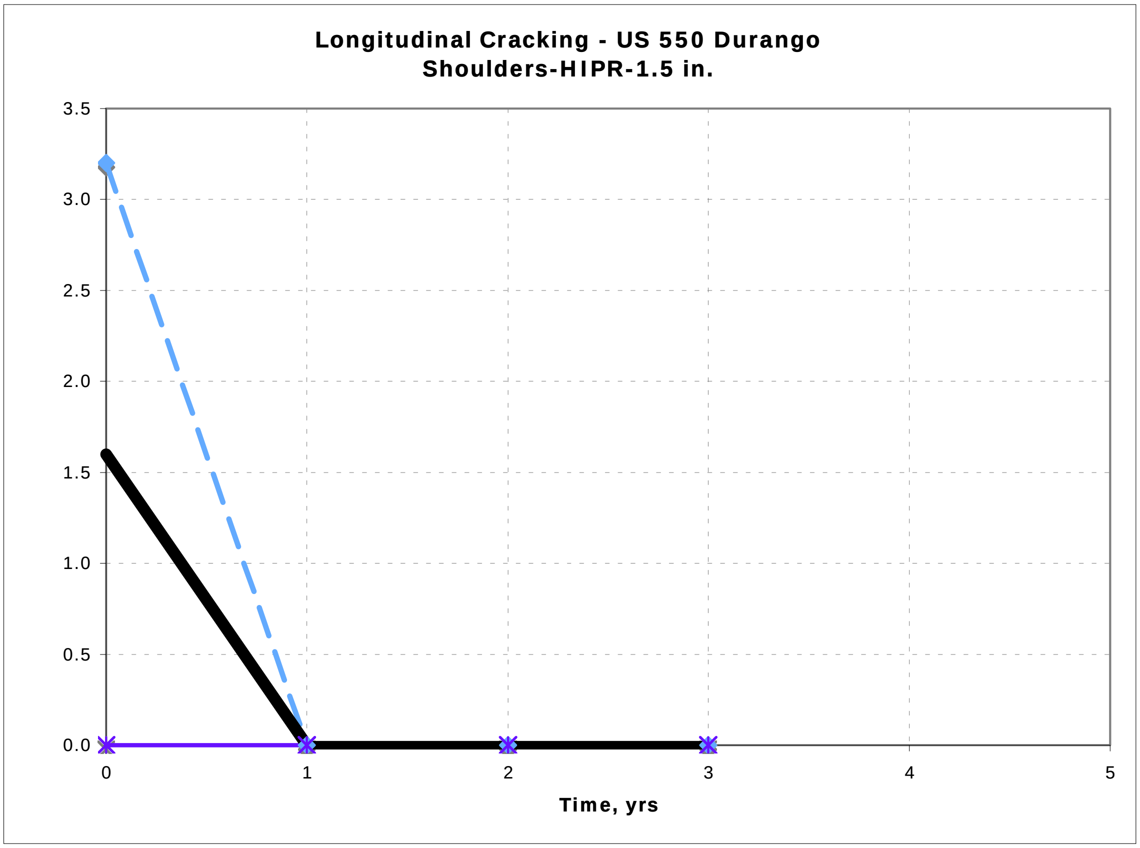


Figure 73. Longitudinal Cracking/500 ft Section for US 550, Durango – Shoulders, HIPR, 1.5 in.

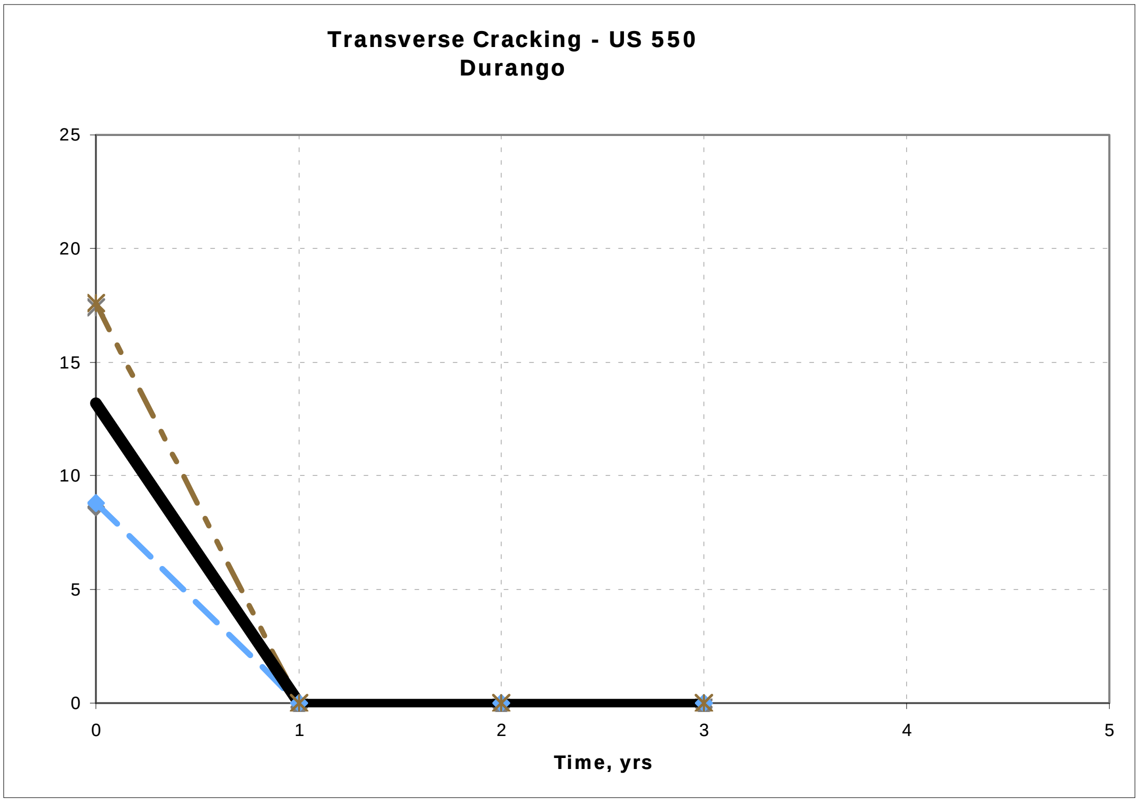


Figure 74. Transverse Cracking/500 ft Section for US 550, Durango – Shoulders, HIPR, 1.5 in.

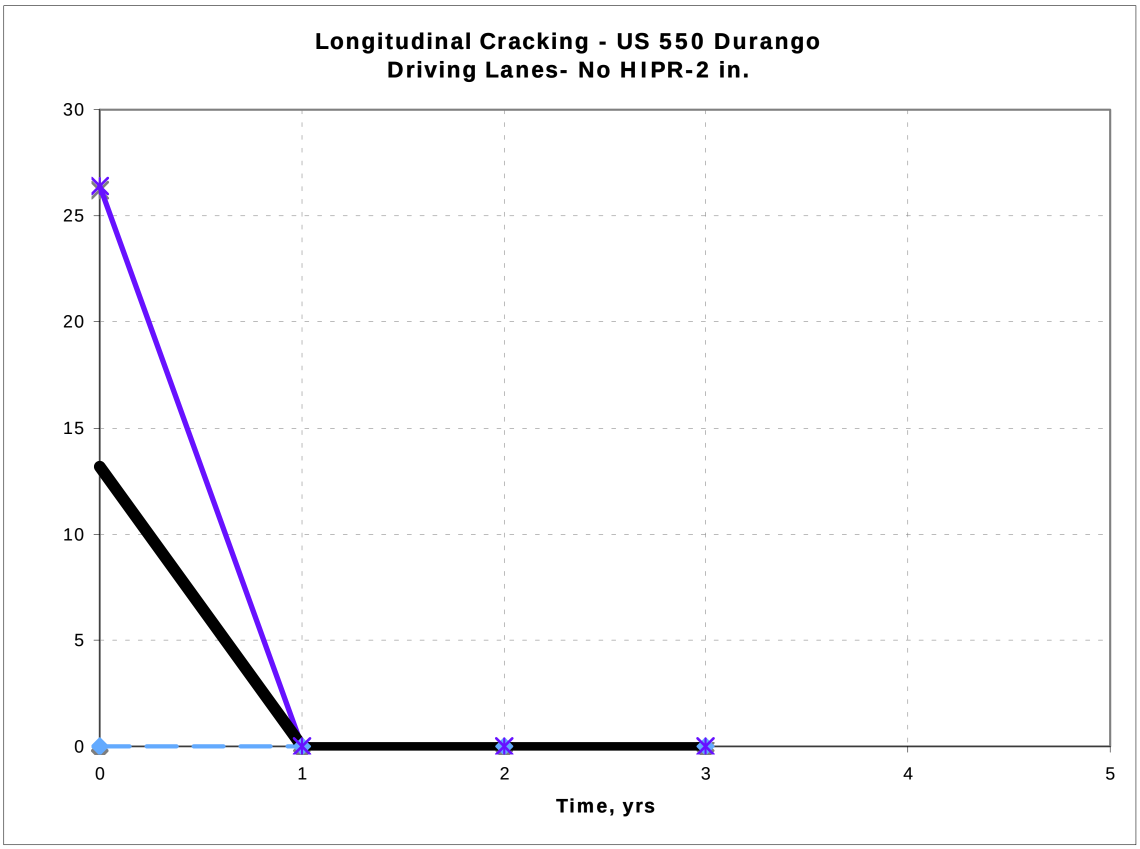


Figure 75. Longitudinal Cracking/500 ft Section for US 550, Durango – Driving Lanes, No HIPR, 2 in.

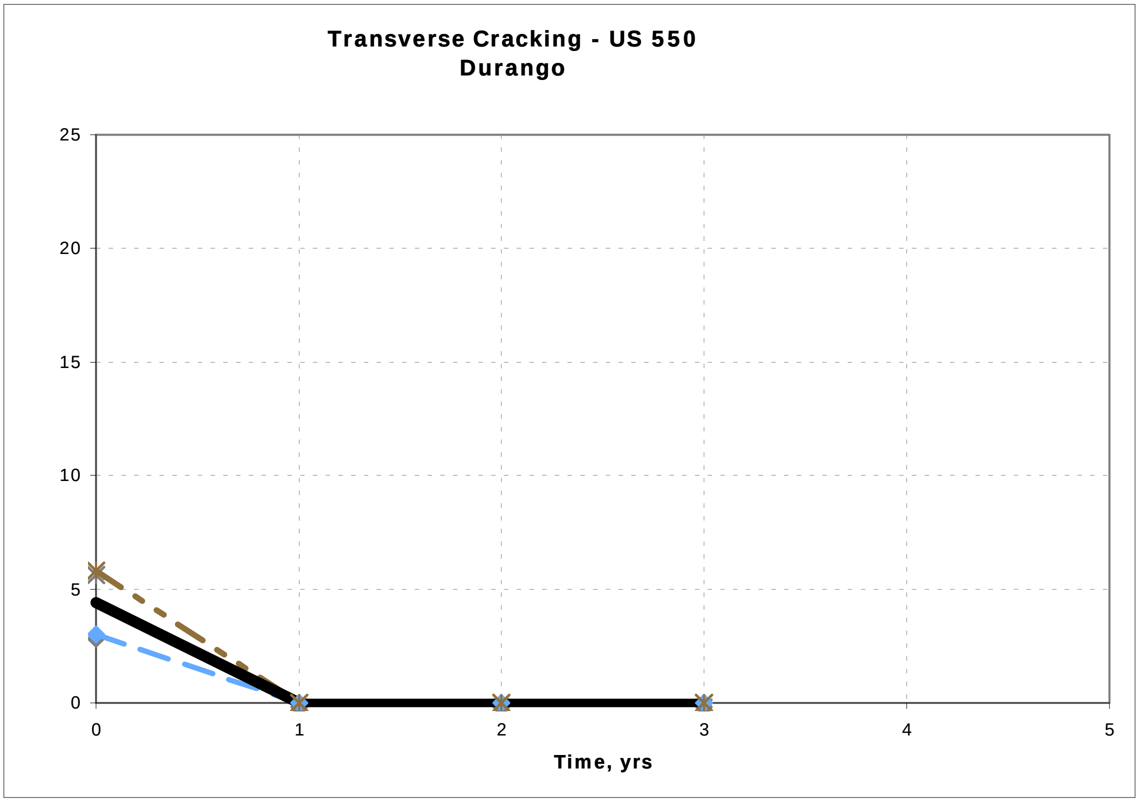


Figure 76. Transverse Cracking/500 ft Section for US 550, Durango – Driving Lanes, No HIPR, 2 in.

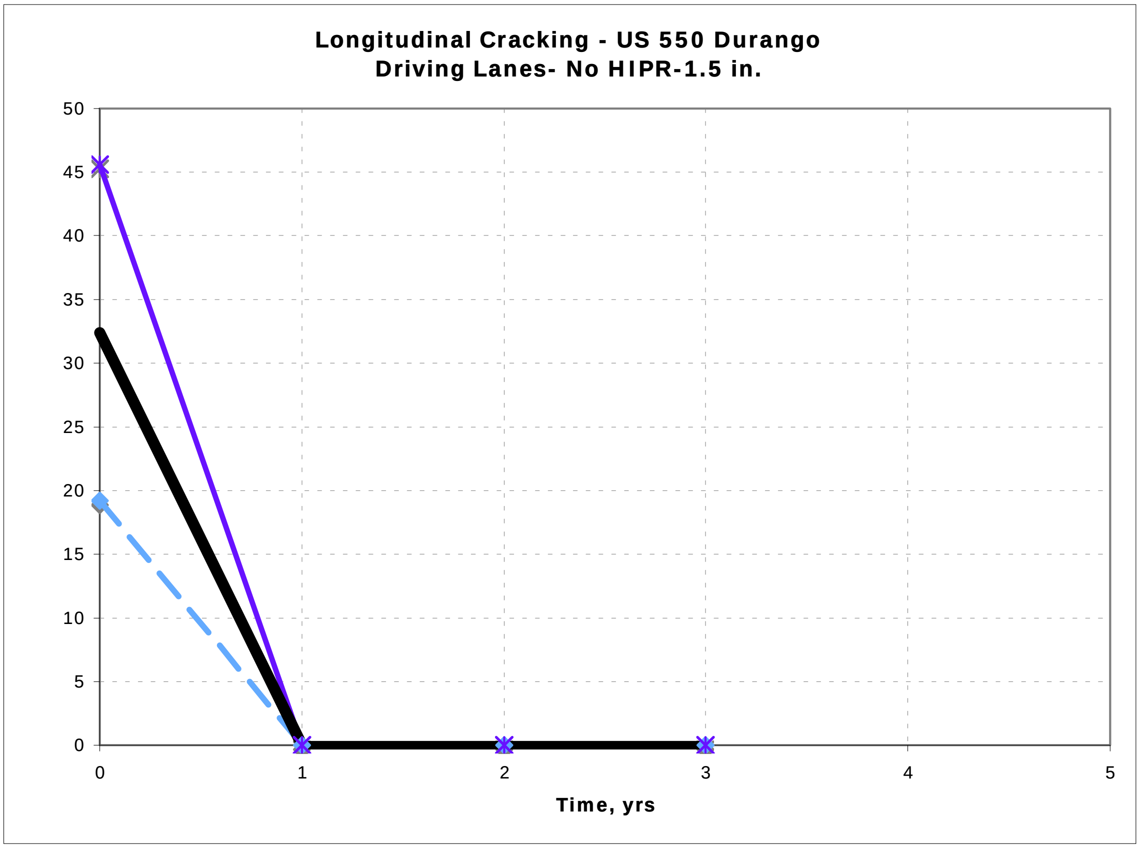


Figure 77. Longitudinal Cracking/500 ft Section for US 550, Durango – Driving Lanes, No HIPR, 1.5 in.

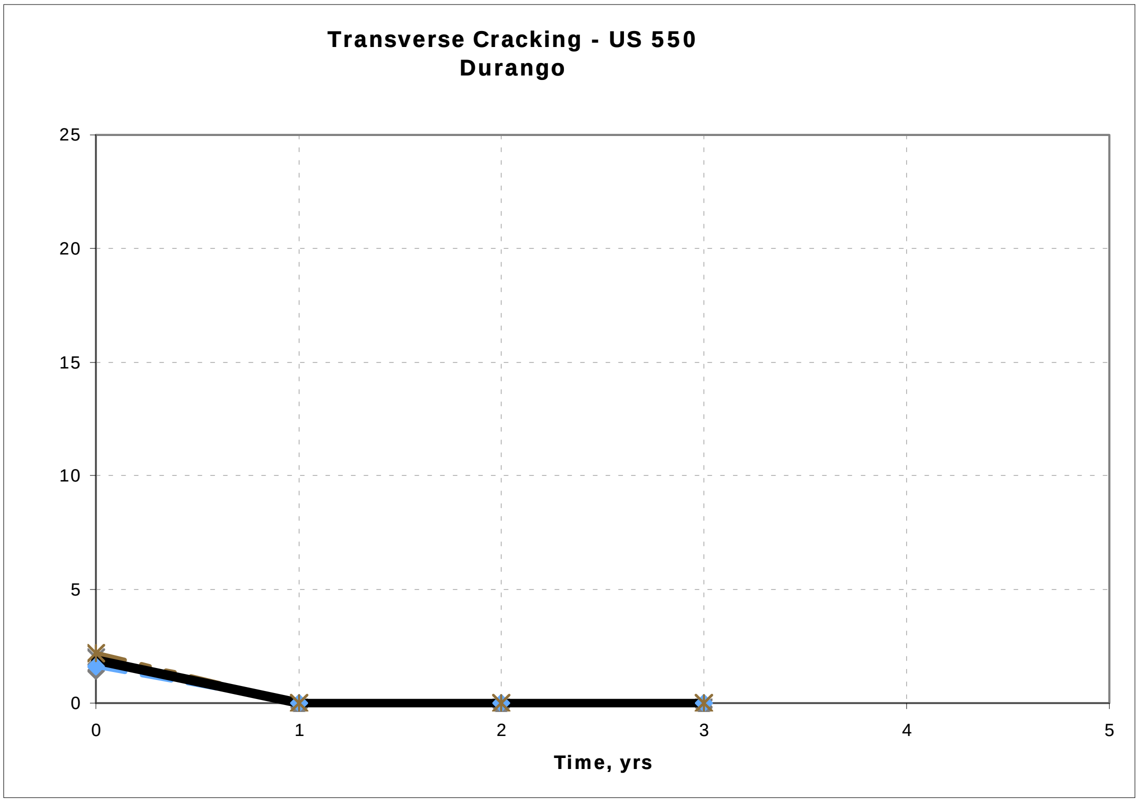


Figure 78. Transverse Cracking/500 ft Section for US 550, Durango – Driving Lanes, No HIPR, 1.5 in.

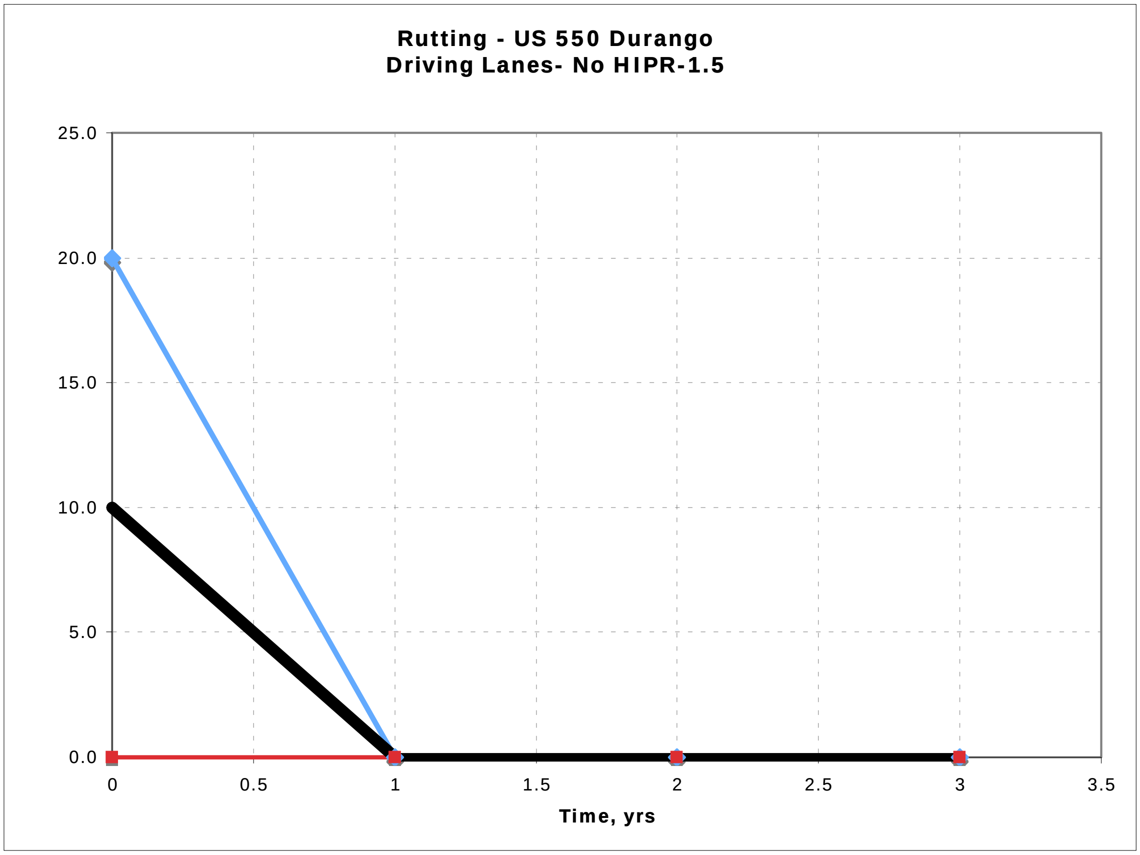


Figure 79. Rutting/500 ft Section for US 550, Durango – Driving Lanes, No HIPR, 1.5 in.

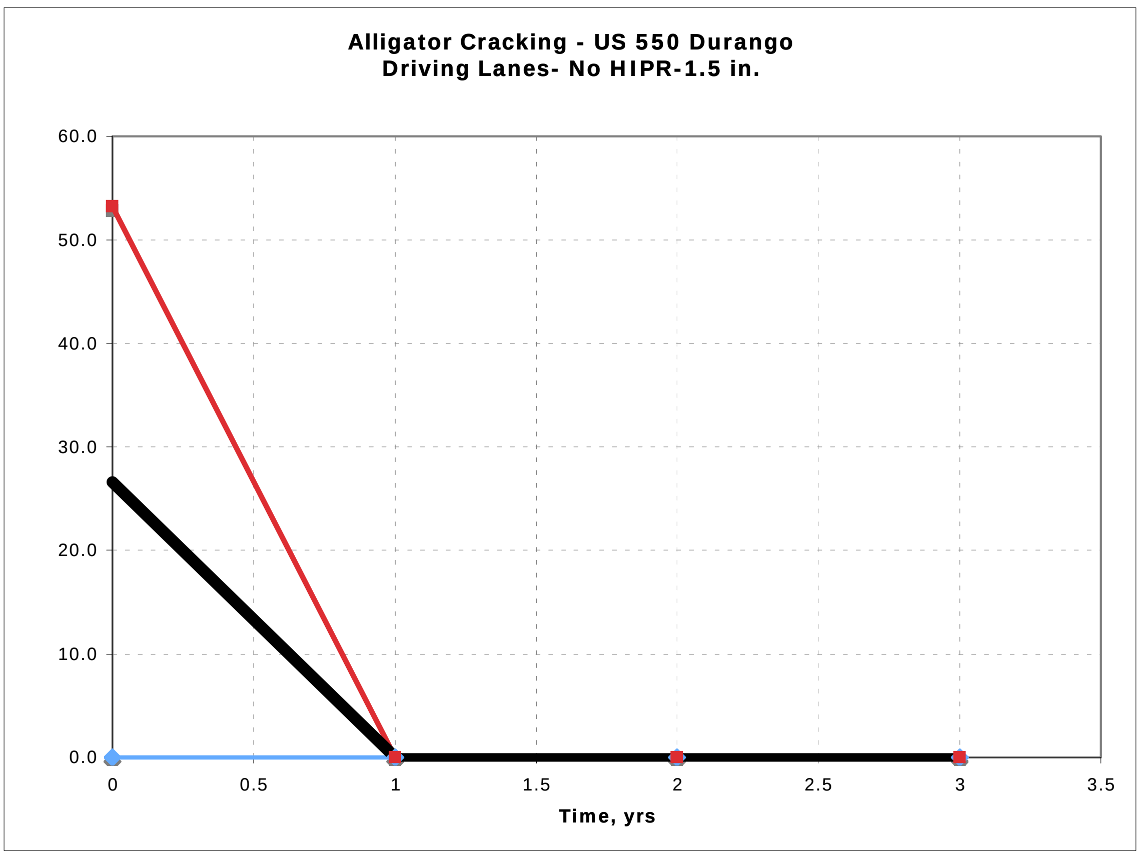


Figure 80. Alligator Cracking/500 ft Section for US 550, Durango – Driving Lanes, No HIPR, 1.5 in.

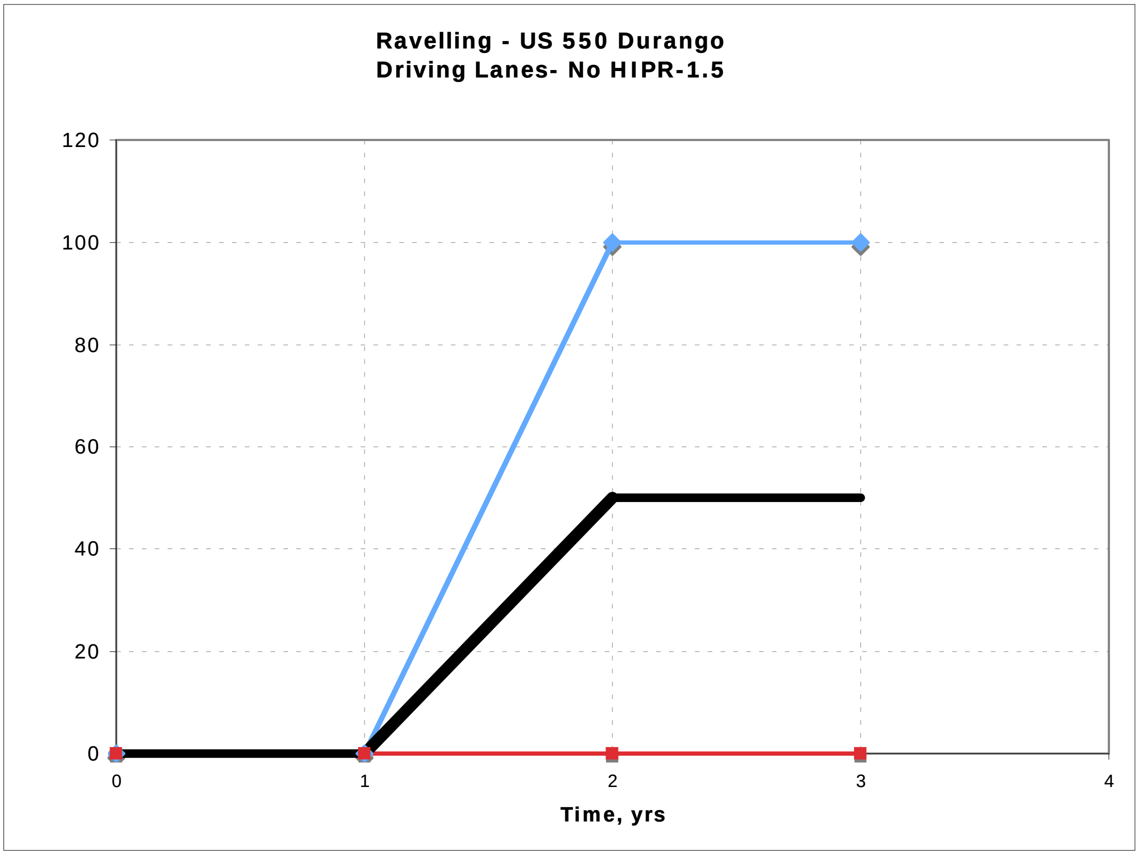


Figure 81. Ravelling/500 ft Section for US 550, Durango – Driving Lanes, No HIPR, 1.5 in.

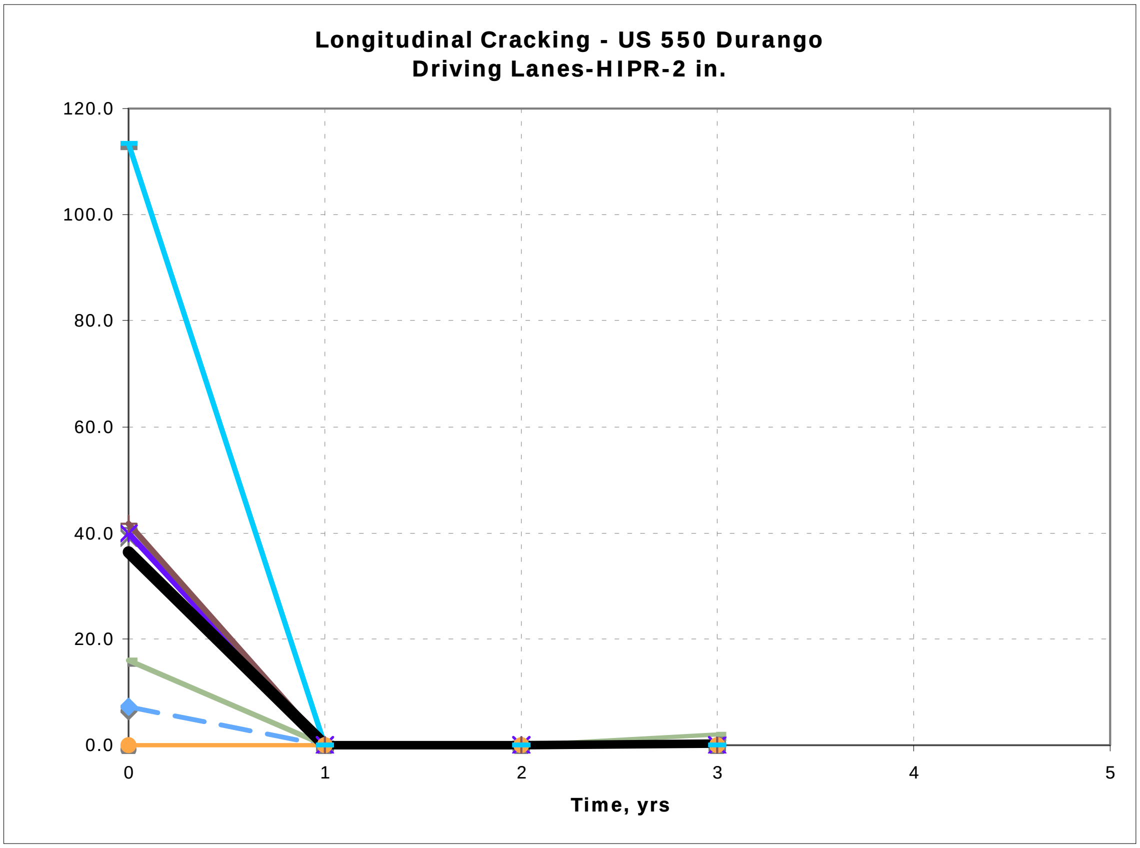


Figure 82. Longitudinal Cracking/500 ft Section for US 550, Durango – Driving Lanes, HIPR, 2 in.

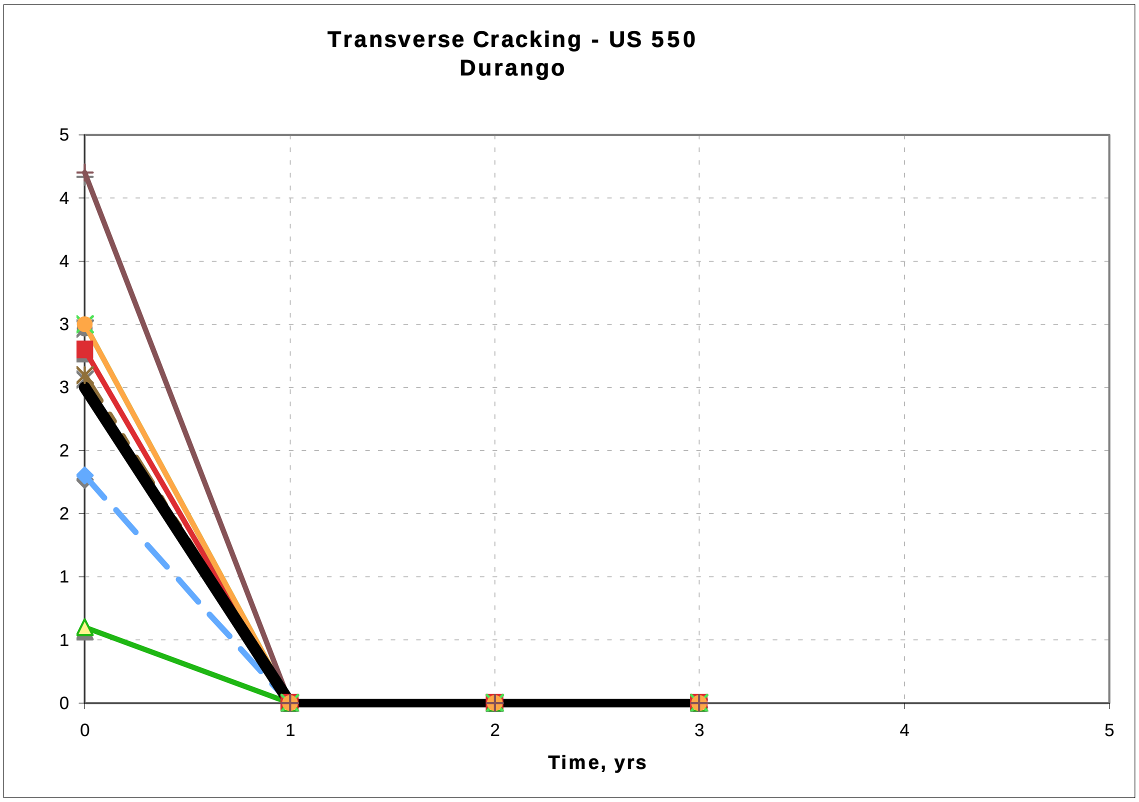


Figure 83. Transverse Cracking/500 ft Section for US 550, Durango – Driving Lanes, HIPR, 2 in.

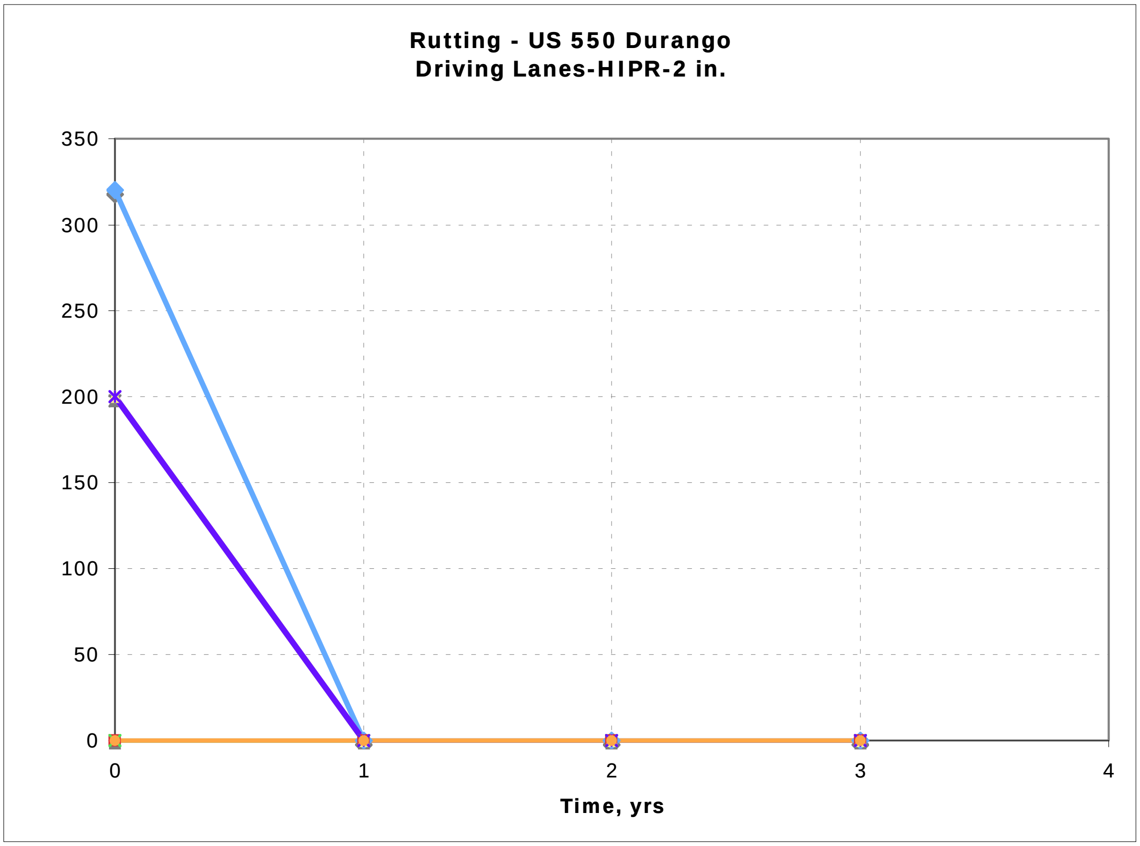


Figure 84. Rutting/500 ft Section for US 550, Durango – Driving Lanes, HIPR, 2 in.

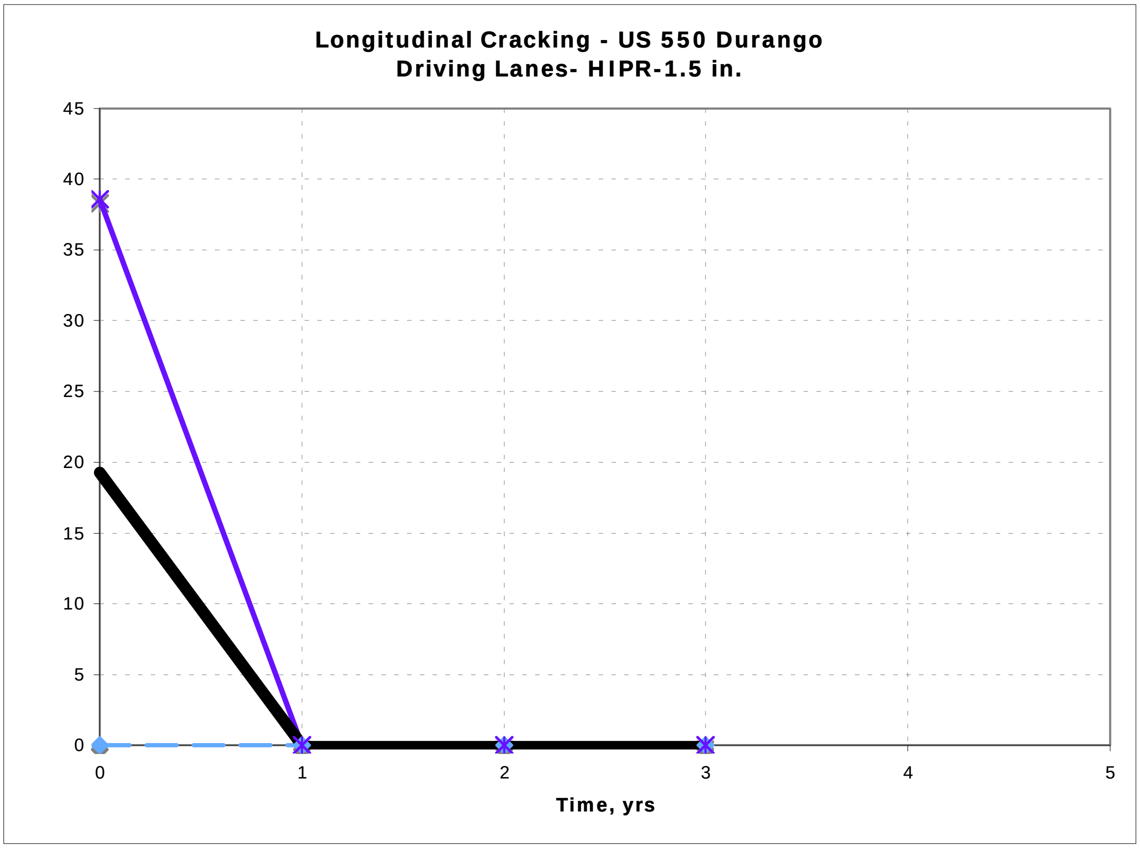


Figure 85. Longitudinal Cracking/500 ft Section for US 550, Durango – Driving Lanes, HIPR, 1.5 in.

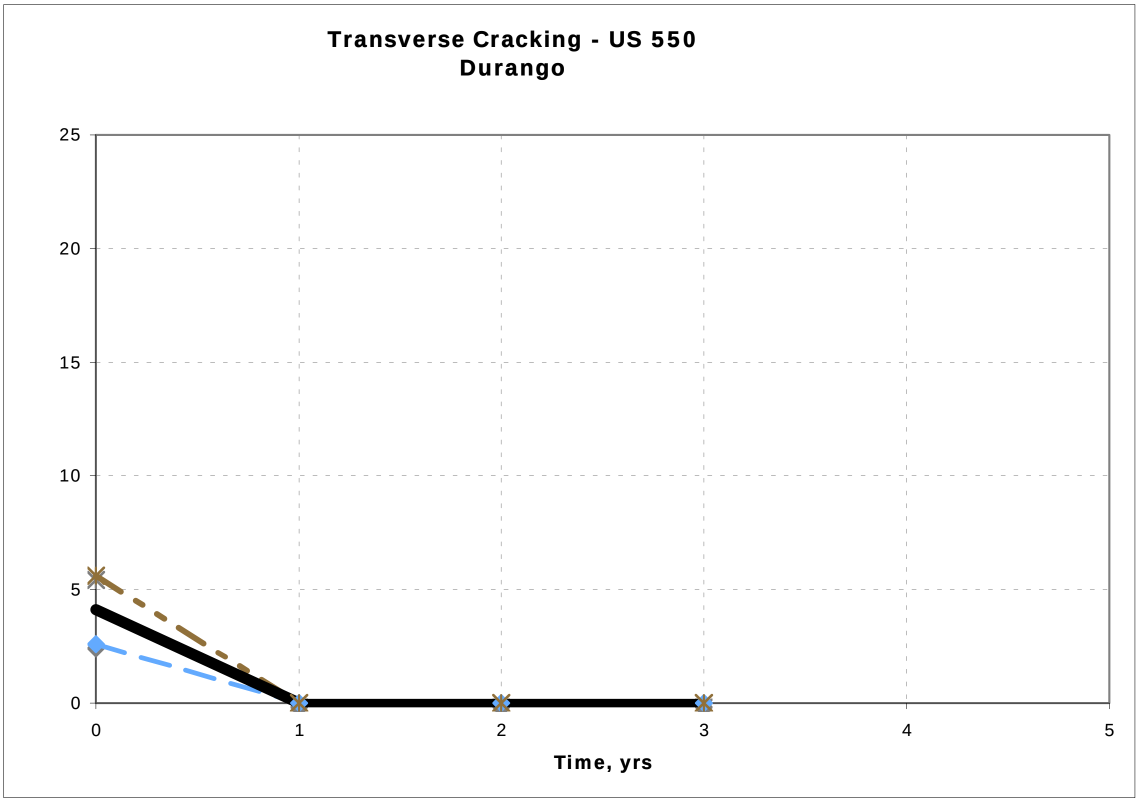


Figure 86. Transverse Cracking/500 ft Section for US 550, Durango – Driving Lanes, HIPR, 1.5 in.

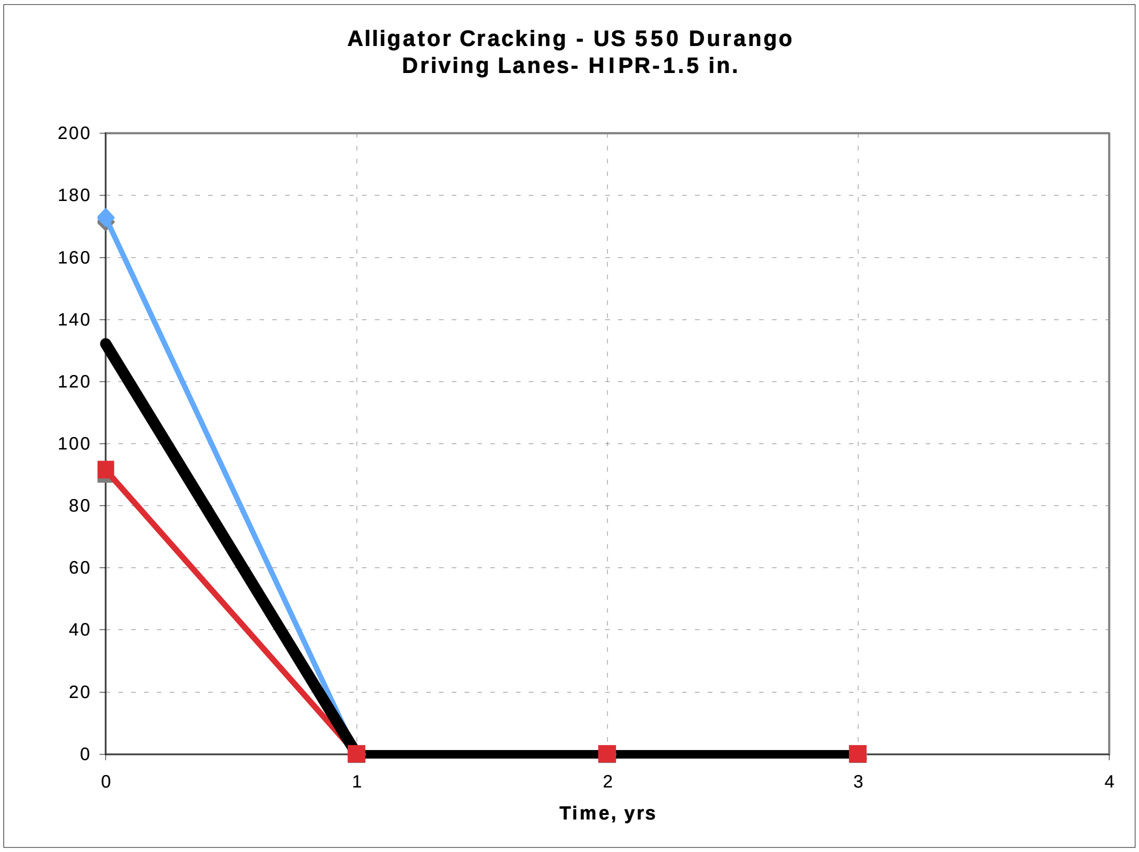


Figure 87. Alligator Cracking/500 ft Section for US 550, Durango – Driving Lanes, HIPR, 1.5 in.

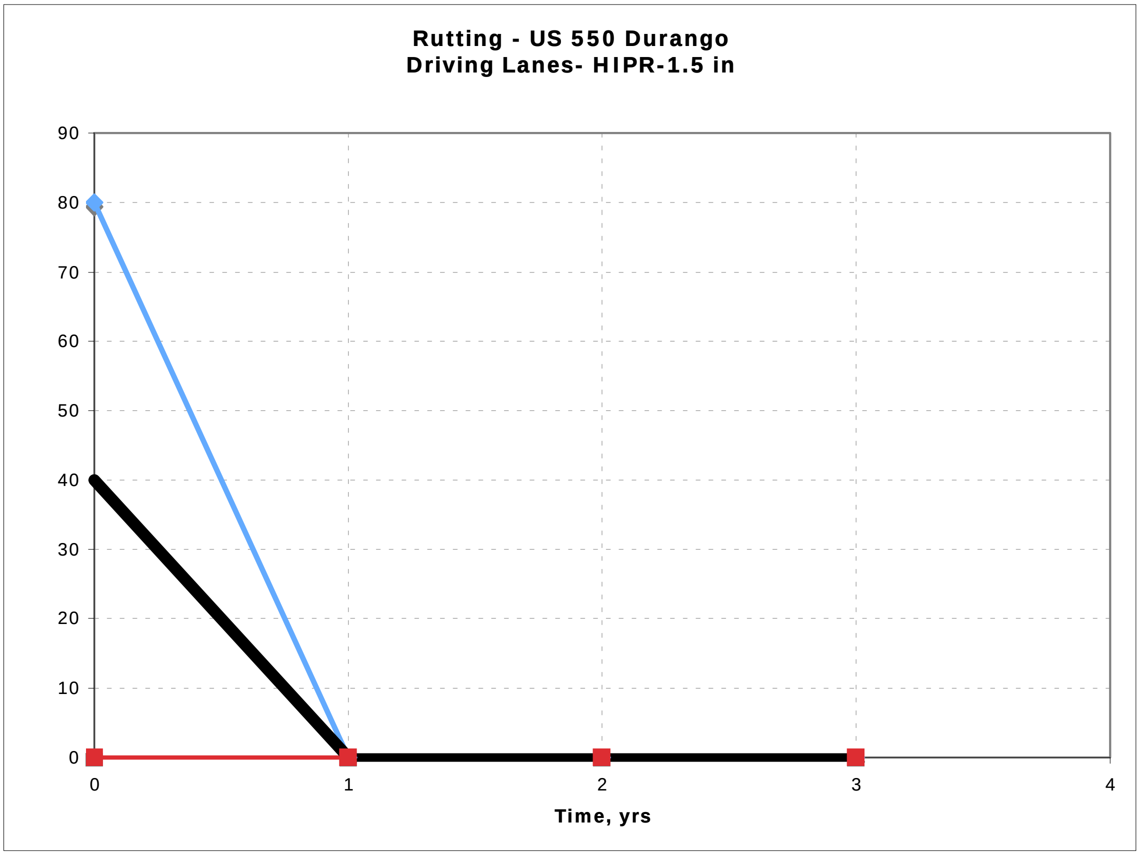


Figure 88. Rutting/500 ft Section for US 550, Durango – Driving Lanes, HIPR, 1.5 in.

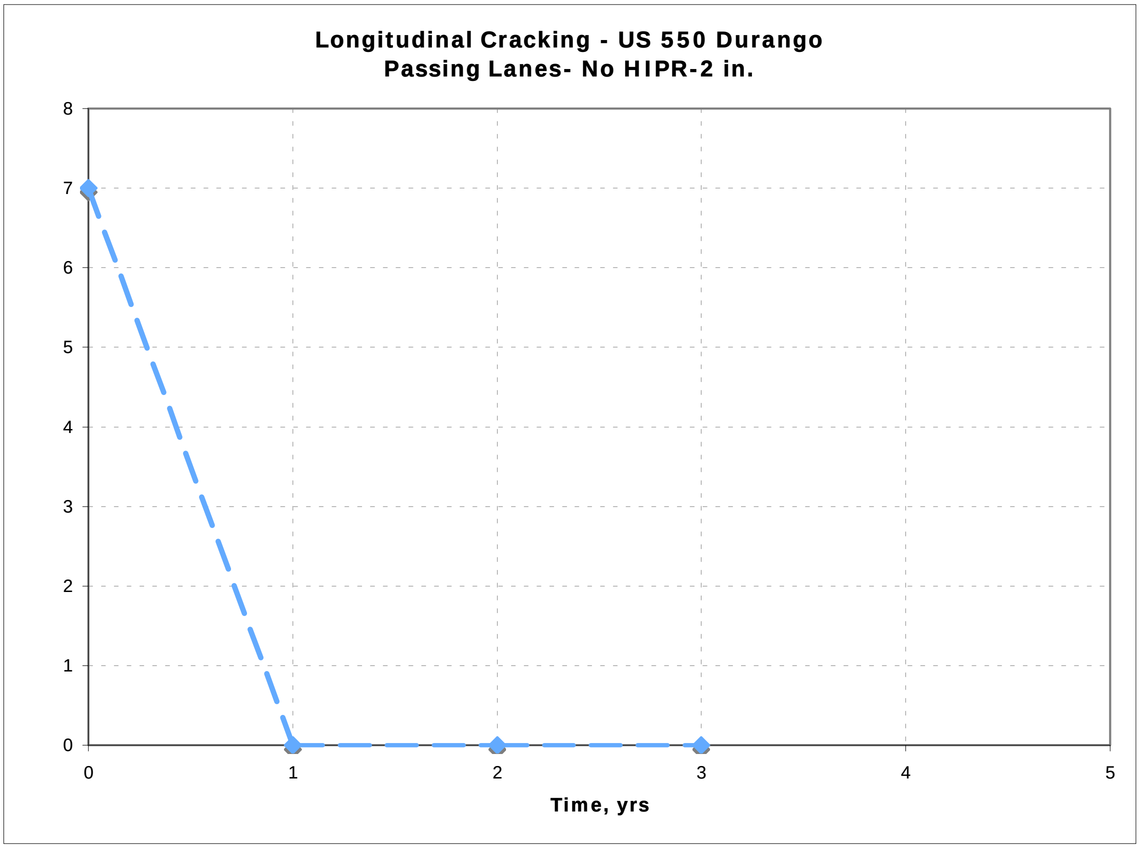


Figure 89. Longitudinal Cracking/500 ft Section for US 550, Durango – Passing Lane, No HIPR, 2 in.

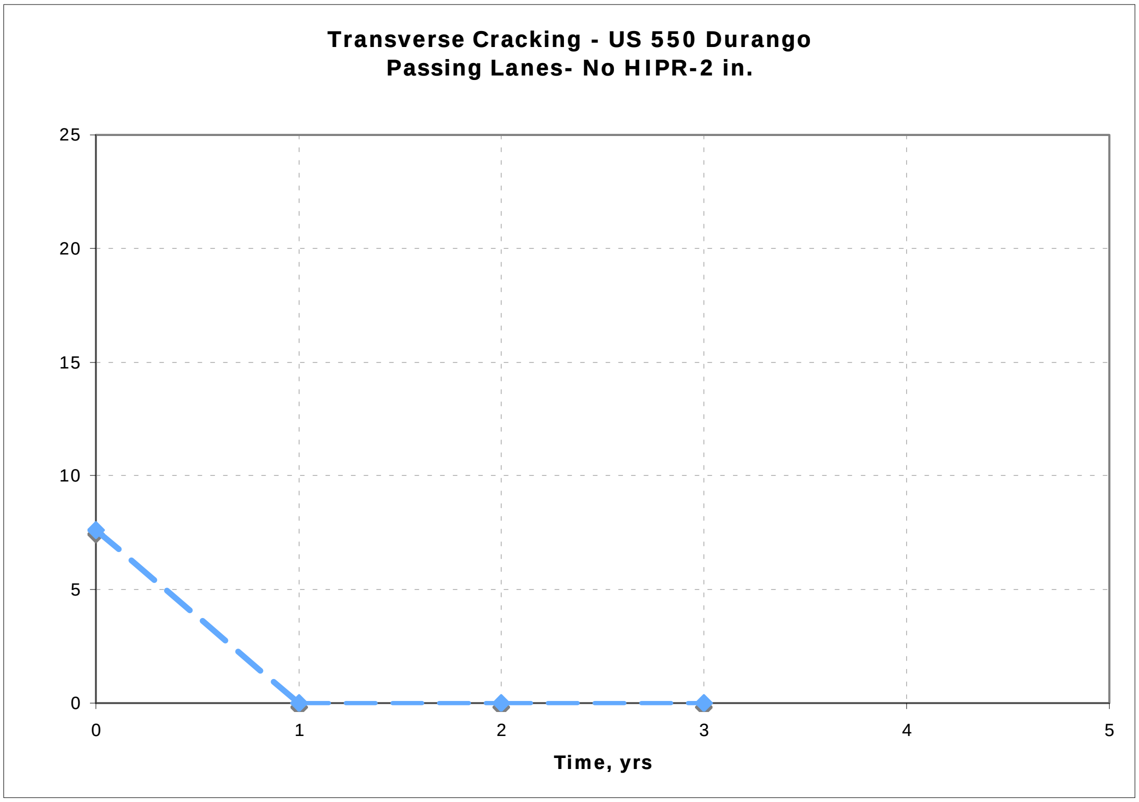


Figure 90. Transverse Cracking/500 ft Section for US 550, Durango – Passing Lane, No HIPR, 2 in.

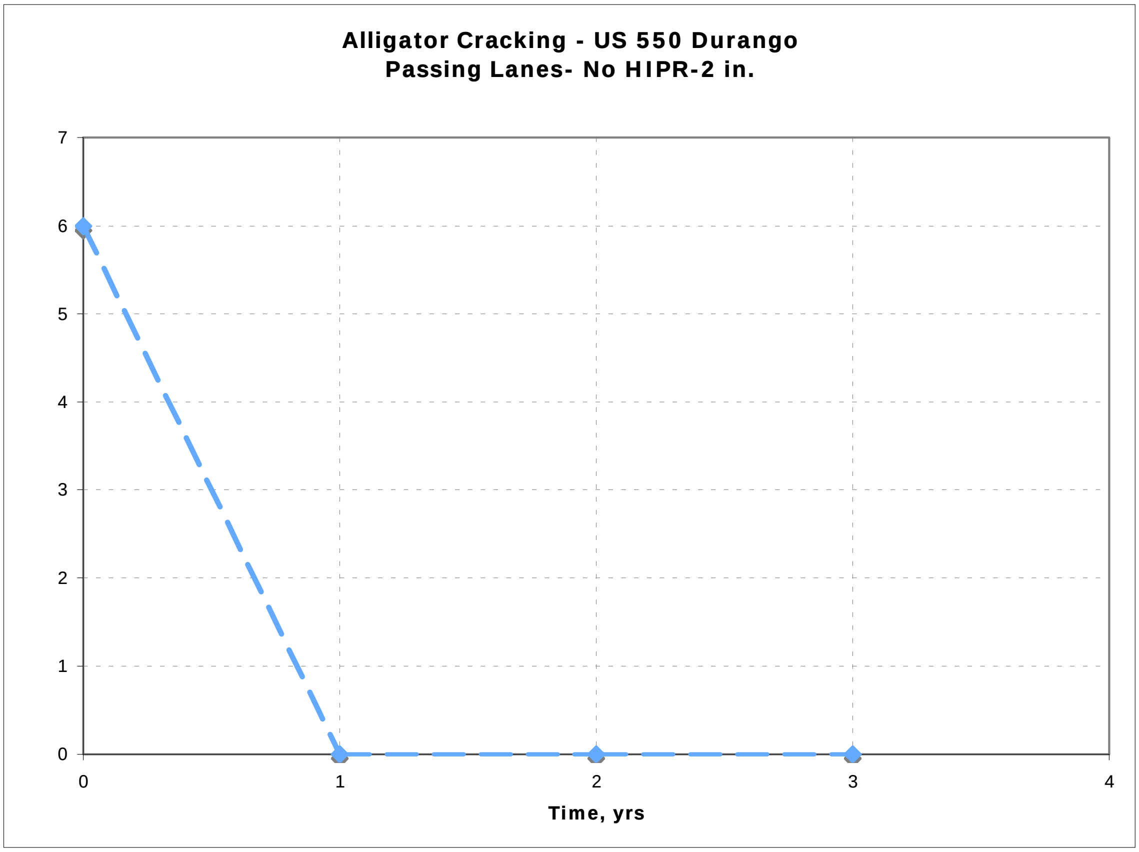


Figure 91. Alligator Cracking/500 ft Section for US 550, Durango – Passing Lane, No HIPR, 2 in.

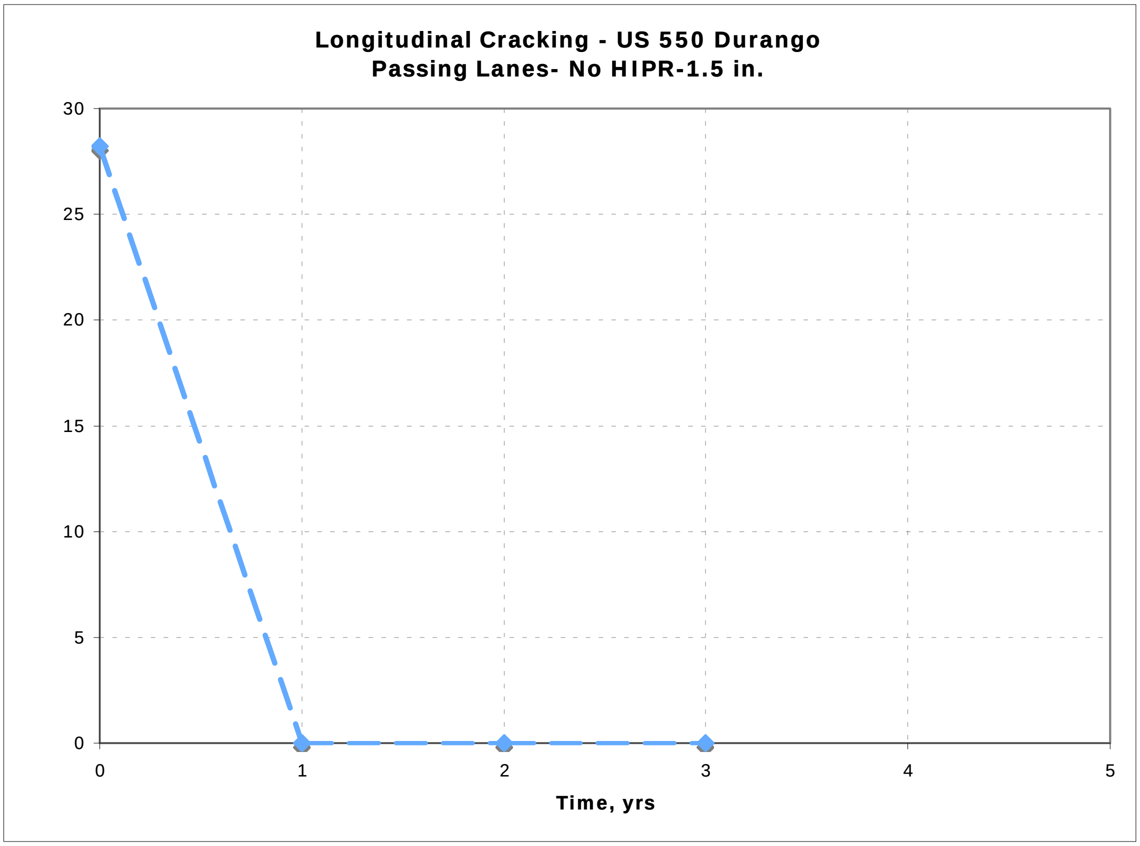


Figure 92. Longitudinal Cracking/500 ft Section for US 550, Durango – Passing Lane, No HIPR, 1.5 in.

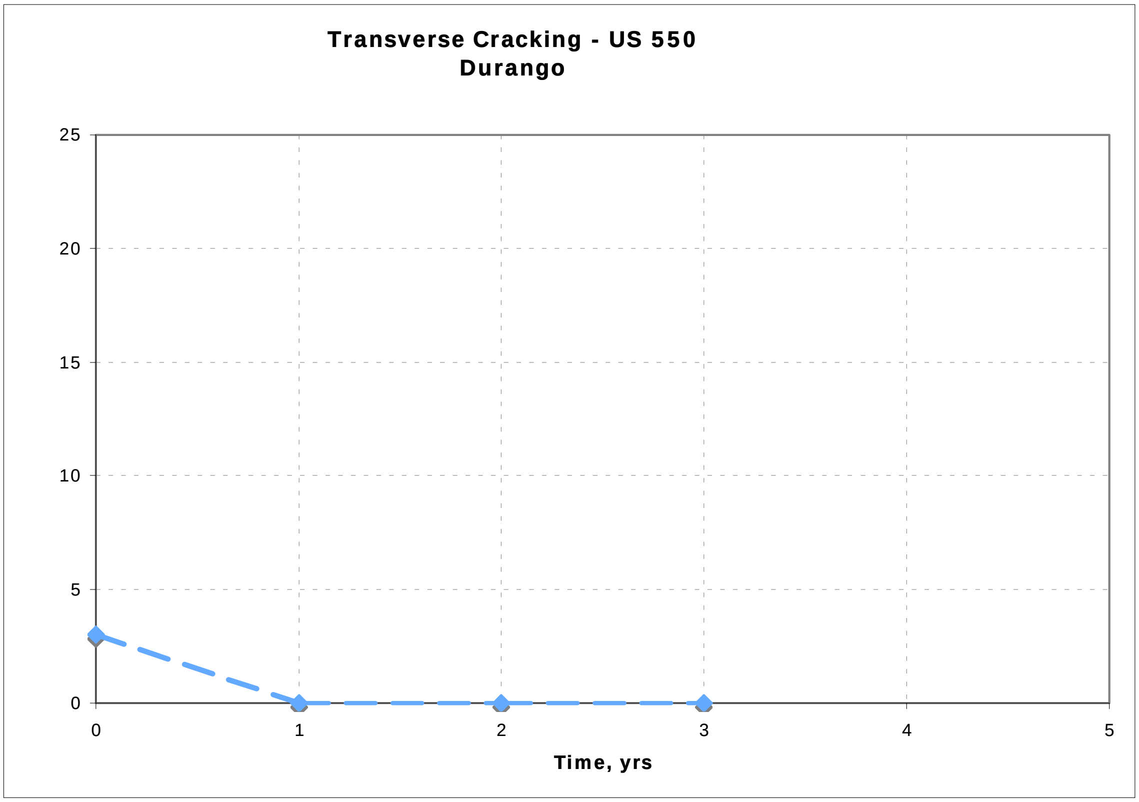


Figure 93. Transverse Cracking/500 ft Section for US 550, Durango – Passing Lane, No HIPR, 1.5 in.

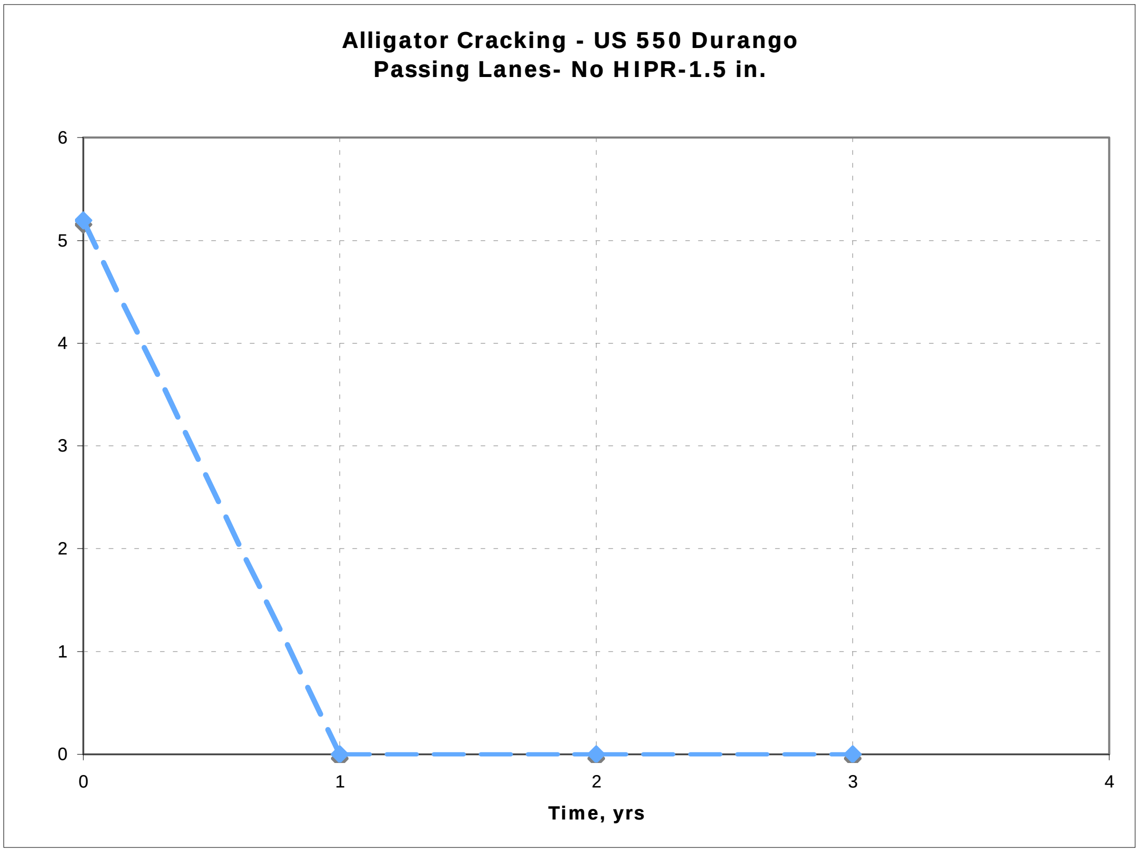


Figure 94. Alligator Cracking/500 ft Section for US 550, Durango – Passing Lane, No HIPR, 1.5 in.

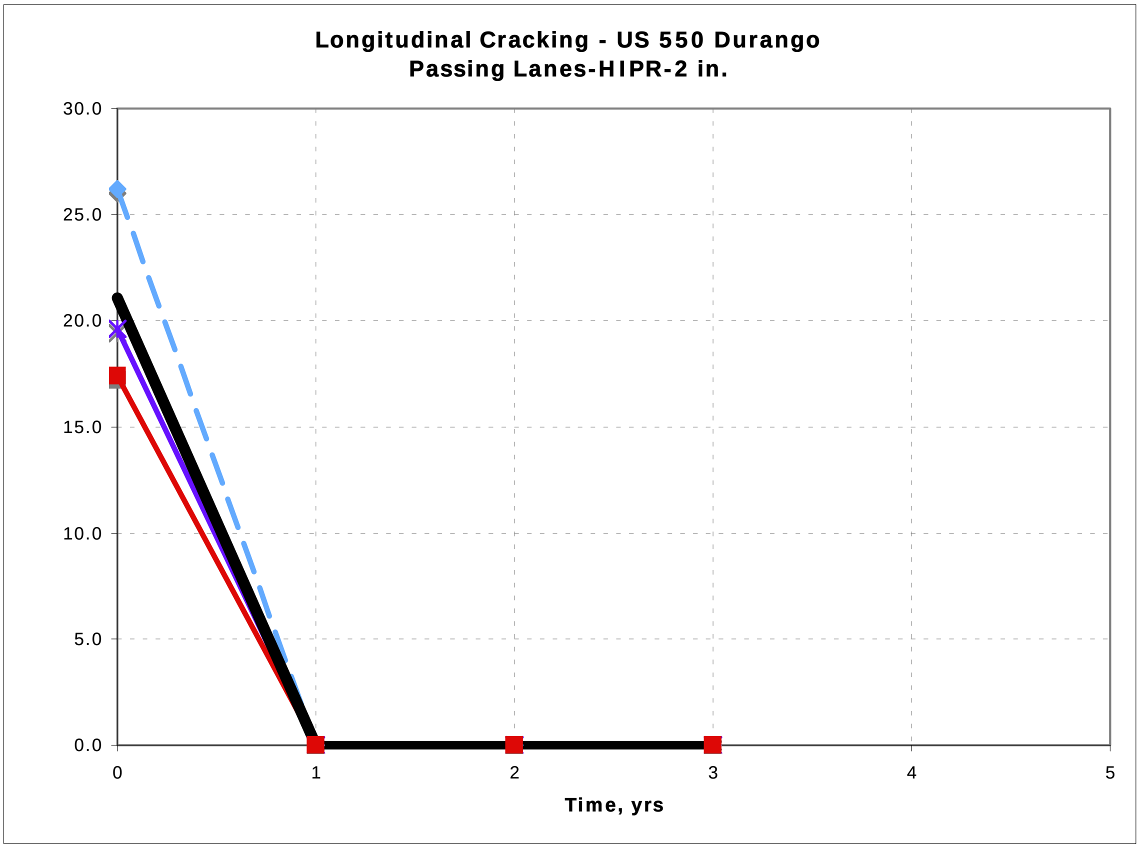


Figure 95. Longitudinal Cracking/500 ft Section for US 550, Durango – Passing Lane, HIPR, 2 in.

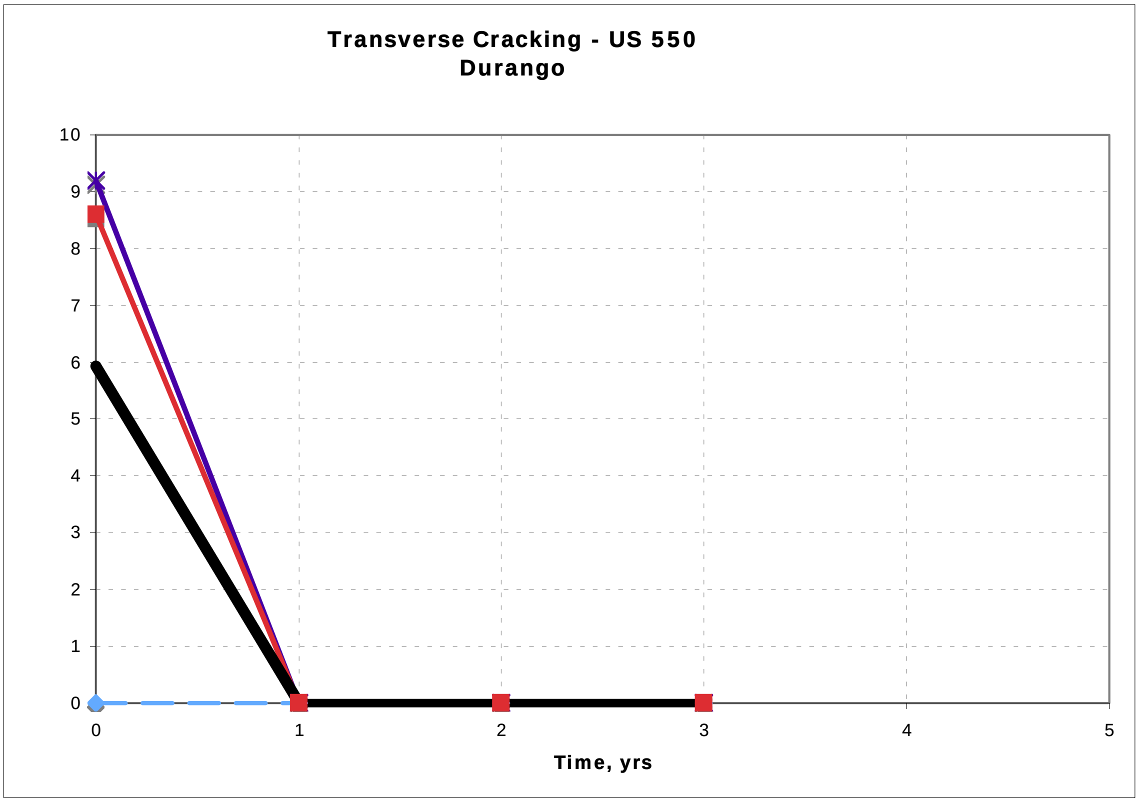


Figure 96. Transverse Cracking/500 ft Section for US 550, Durango – Passing Lane, HIPR, 2 in.

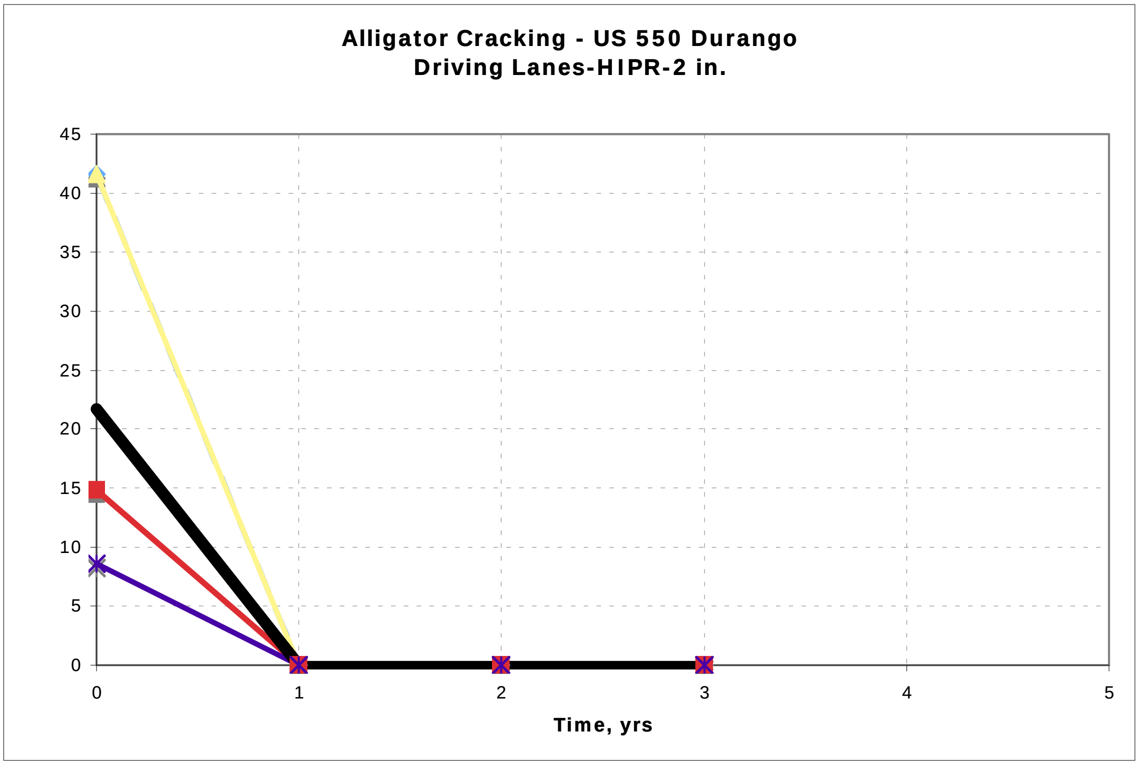


Figure 97. Alligator Cracking/500 ft Section for US 550, Durango – Passing Lane, HIPR, 2 in.

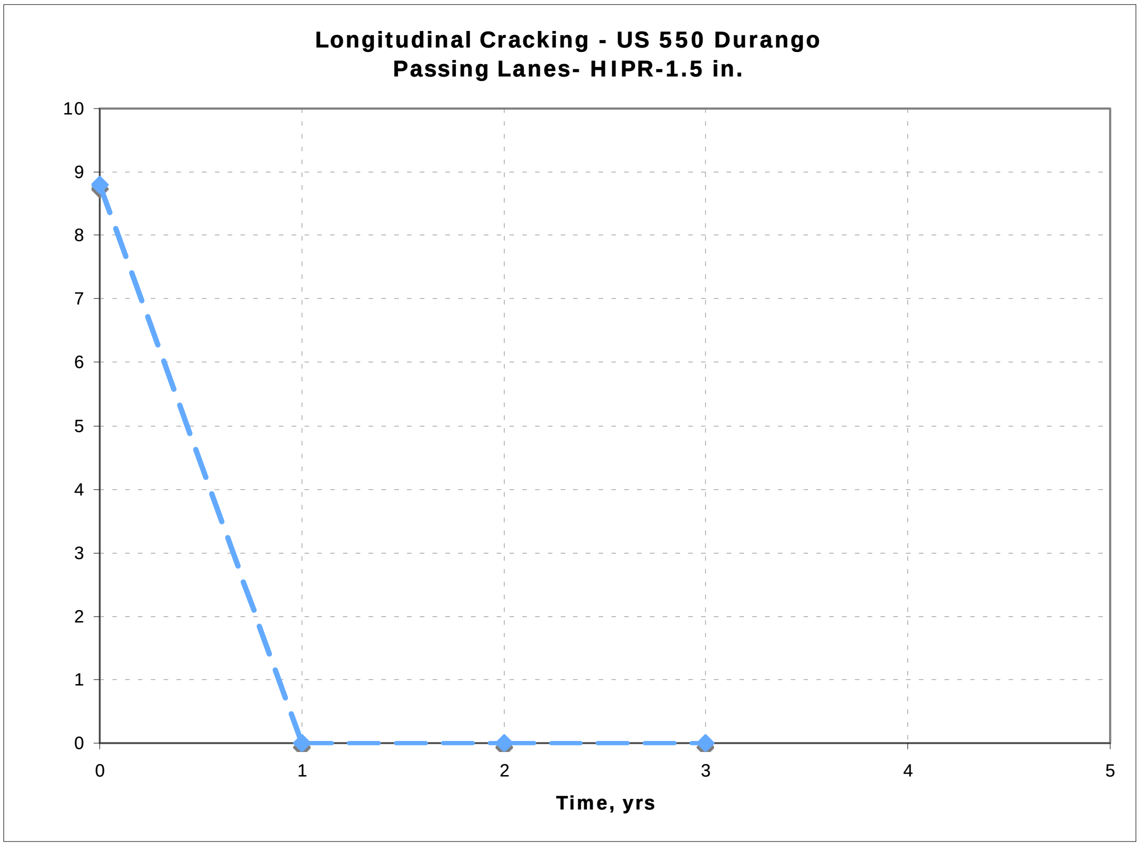


Figure 98. Longitudinal Cracking/500 ft Section for US 550, Durango – Passing Lane, HIPR, 1.5 in.

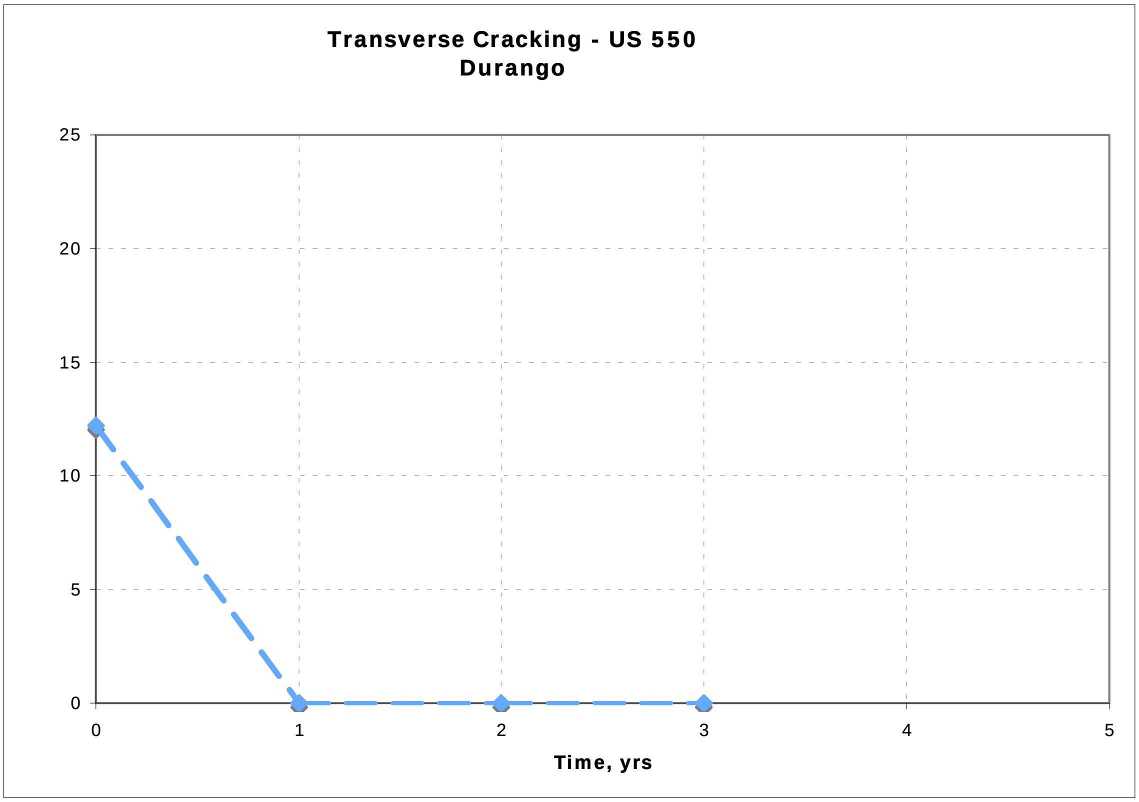


Figure 99. Transverse Cracking/500 ft Section for US 550, Durango – Passing Lane, HIPR, 1.5 in.

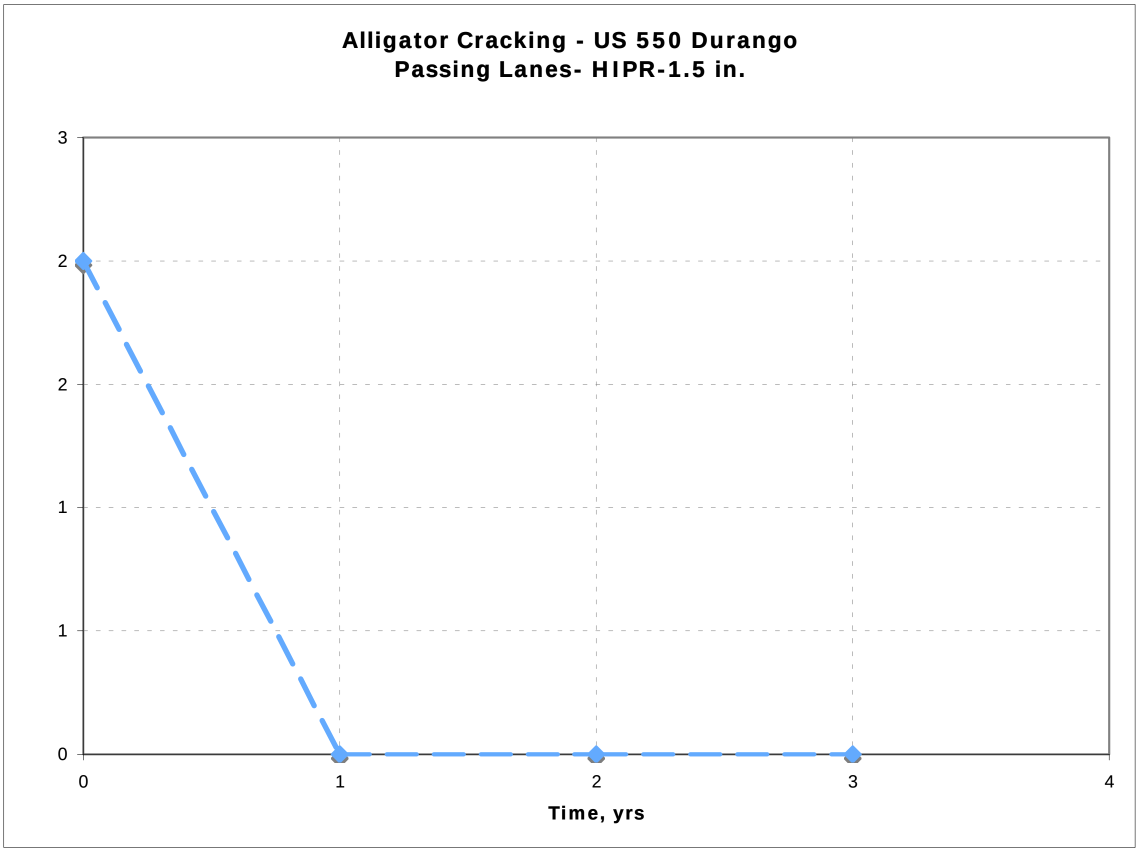


Figure 100. Alligator Cracking/500 ft Section for US 550, Durango – Passing Lane, HIPR, 1.5 in.

4.4.4 US 550, Coal Bank Pass

Four evaluation sections were established on US 550 south of Coal Bank Pass, two in the southbound lane and two sections were monitored in the northbound lane. The results of three years of performance data on these sections and the average are shown in Figures 101 through 107.

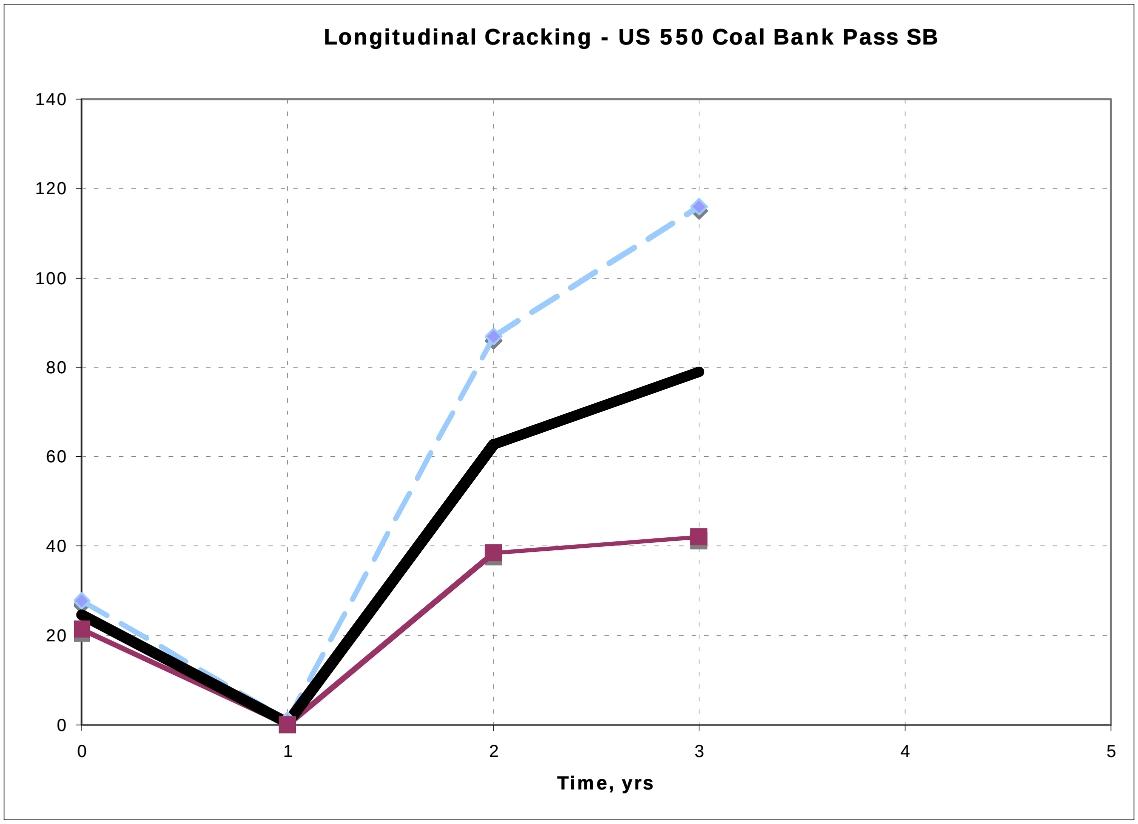


Figure 101. Longitudinal Cracking/500 ft Section for US 550, Coal Bank Pass Southbound

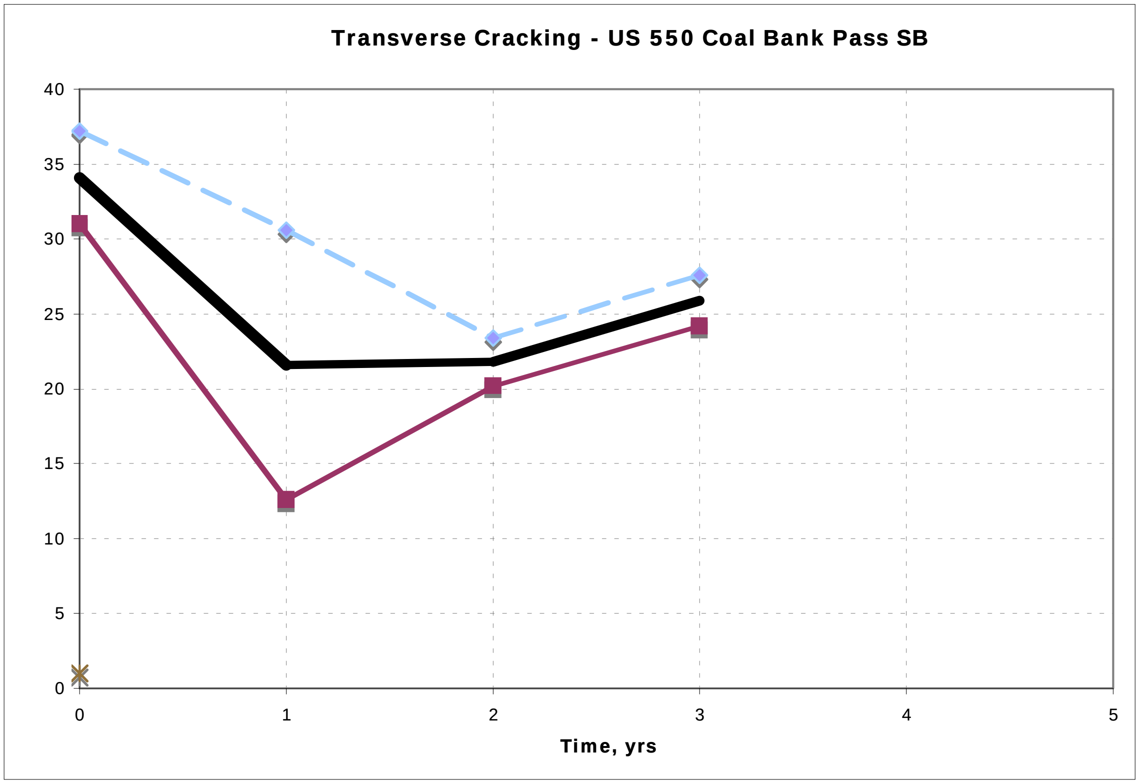


Figure 102. Transverse Cracking/500 ft Section for US 550, Coal Bank Pass Southbound

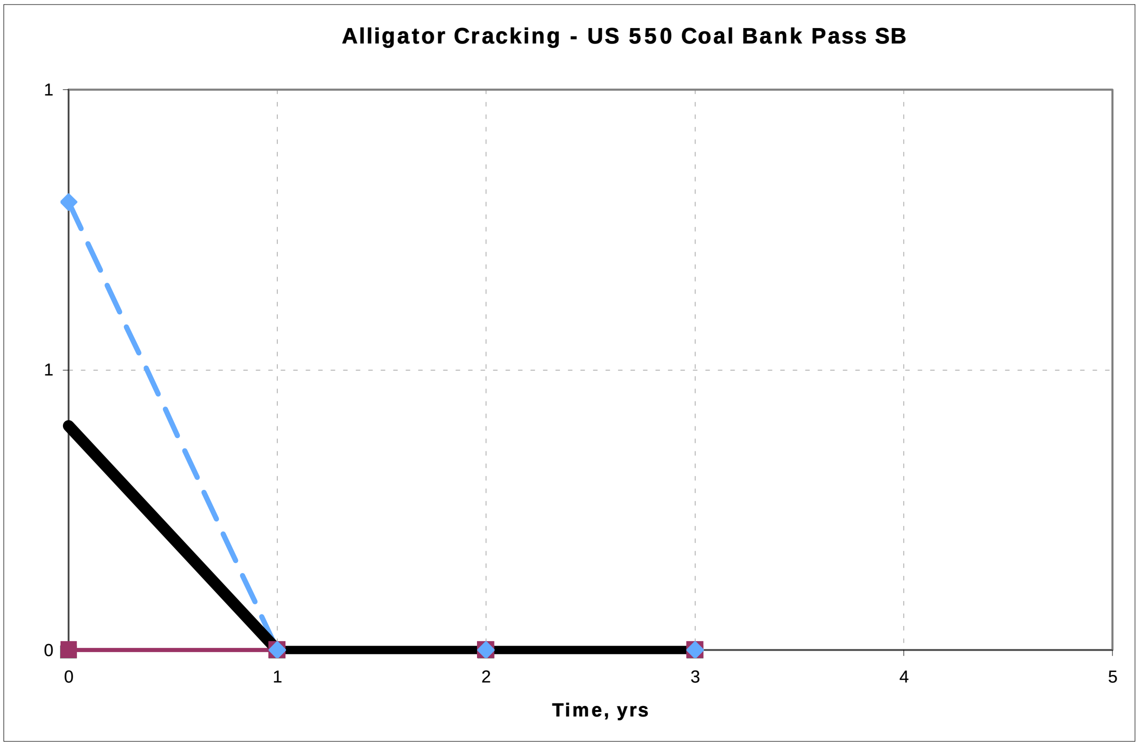


Figure 103. Alligator Cracking/500 ft Section for US 550, Coal Bank Pass Southbound

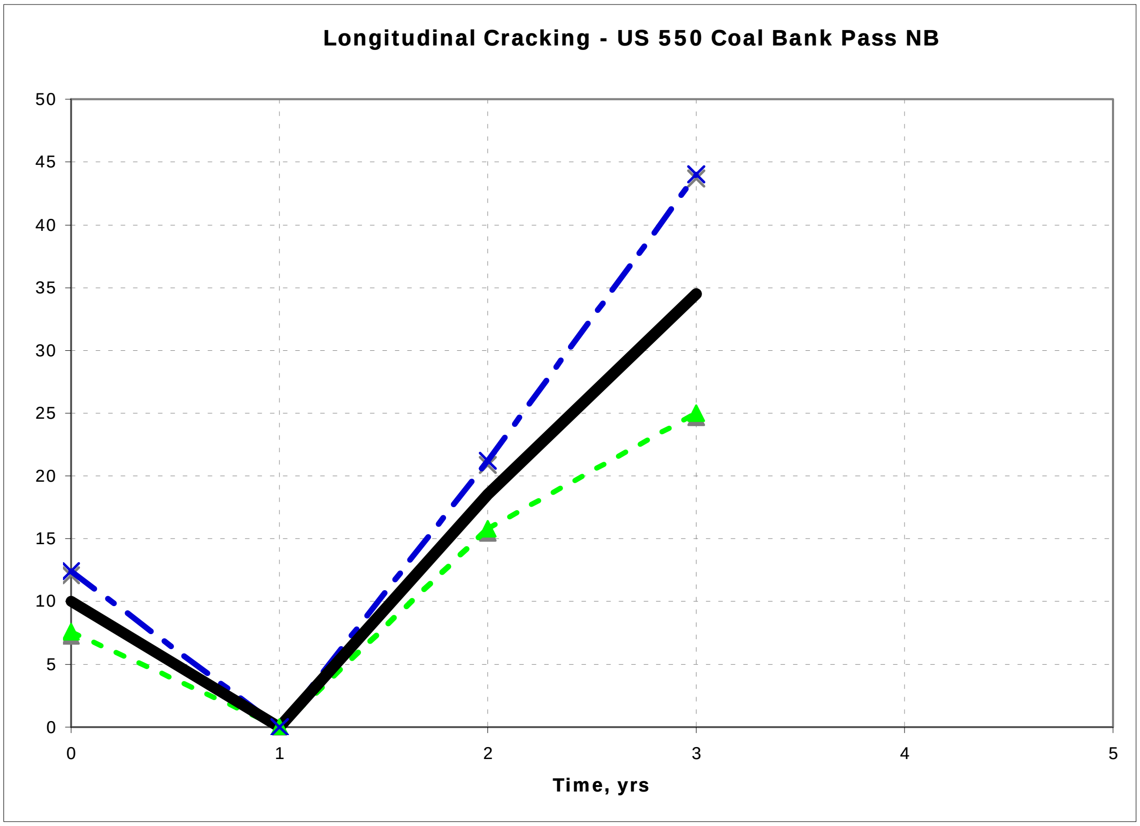


Figure 104. Longitudinal Cracking/500 ft Section for US 550, Coal Bank Pass Northbound

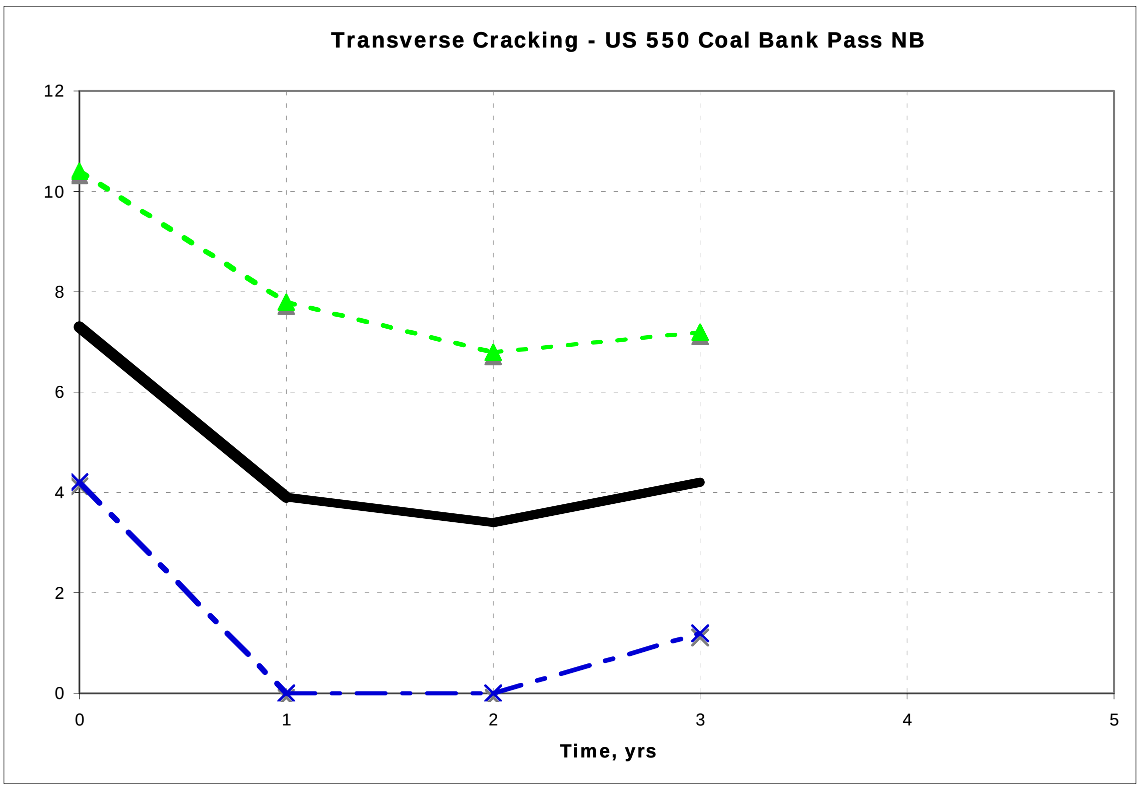


Figure 105. Transverse Cracking/500 ft Section for US 550, Coal Bank Pass Northbound

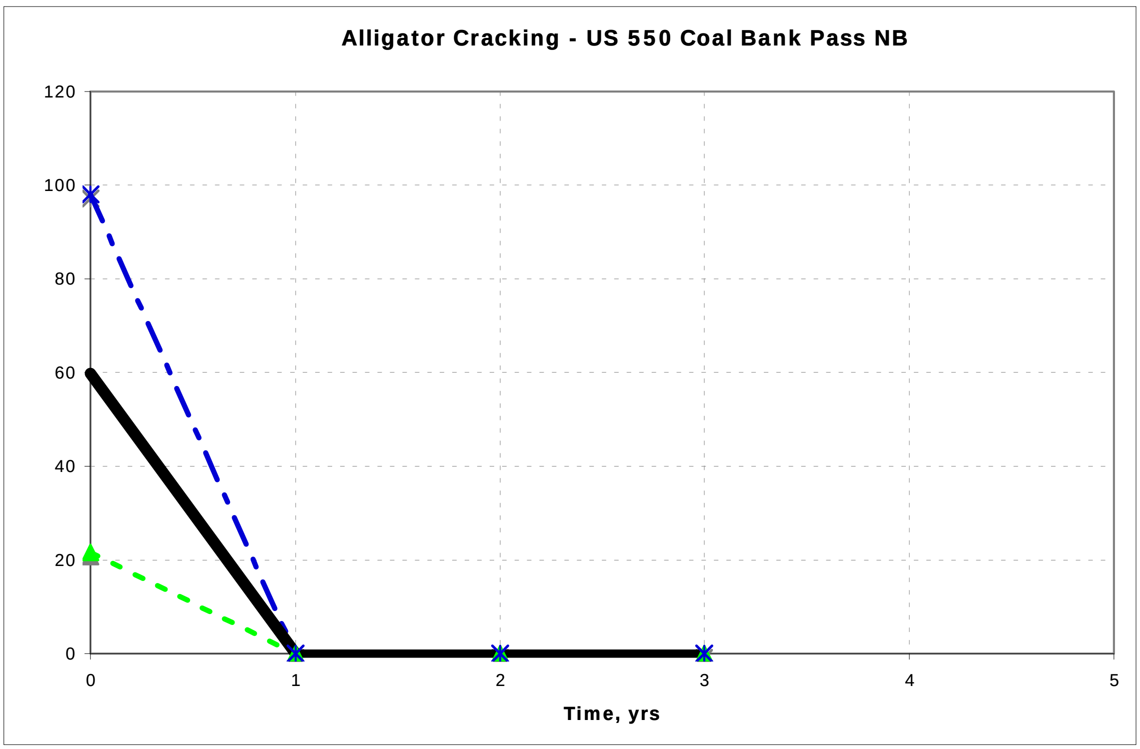


Figure 106. Alligator Cracking/500 ft Section for US 550 Coal Bank Pass Northbound

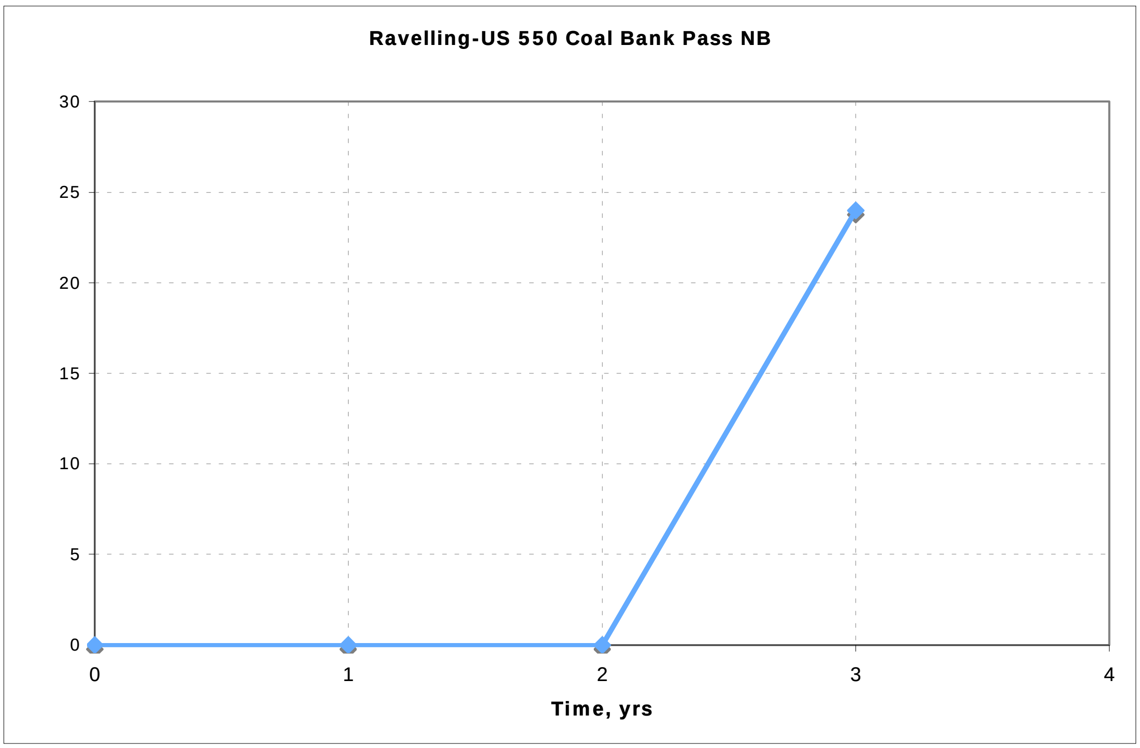


Figure 107. Ravelling/500 ft Section for US 550, Coal Bank Pass Northbound

4.4.5 US 40, Golden

Two evaluation sections were established on US 40, Golden. The results of three years of performance data on both sections and the average are shown in Figures 108 through 110.

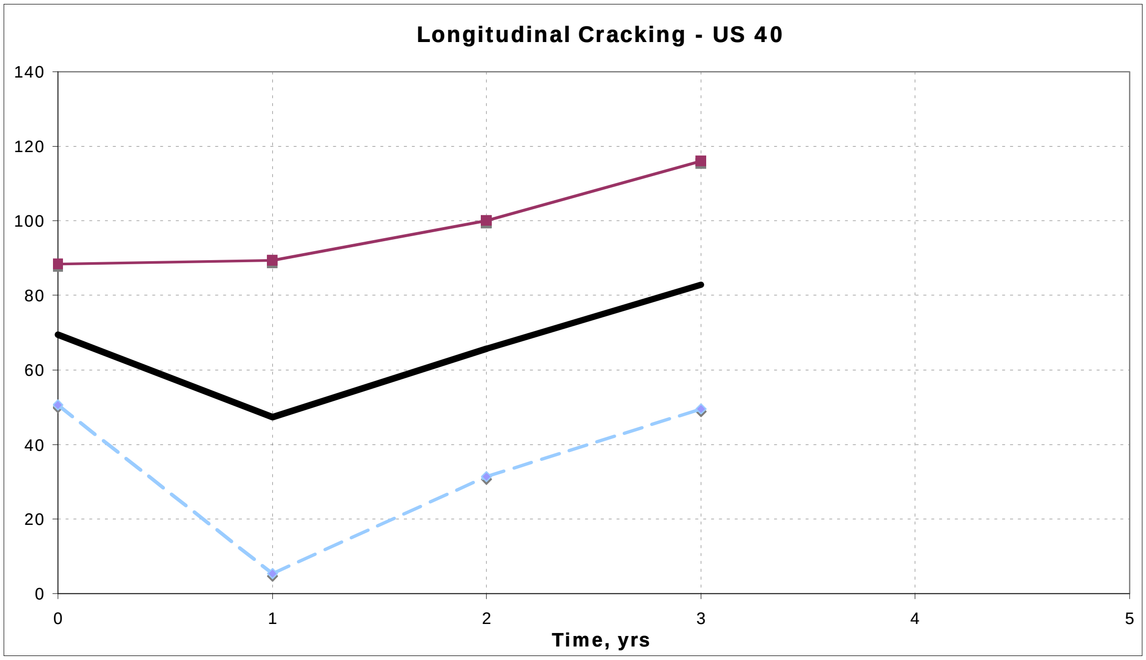


Figure 108. Longitudinal Cracking/500 ft Section for US 40, Golden

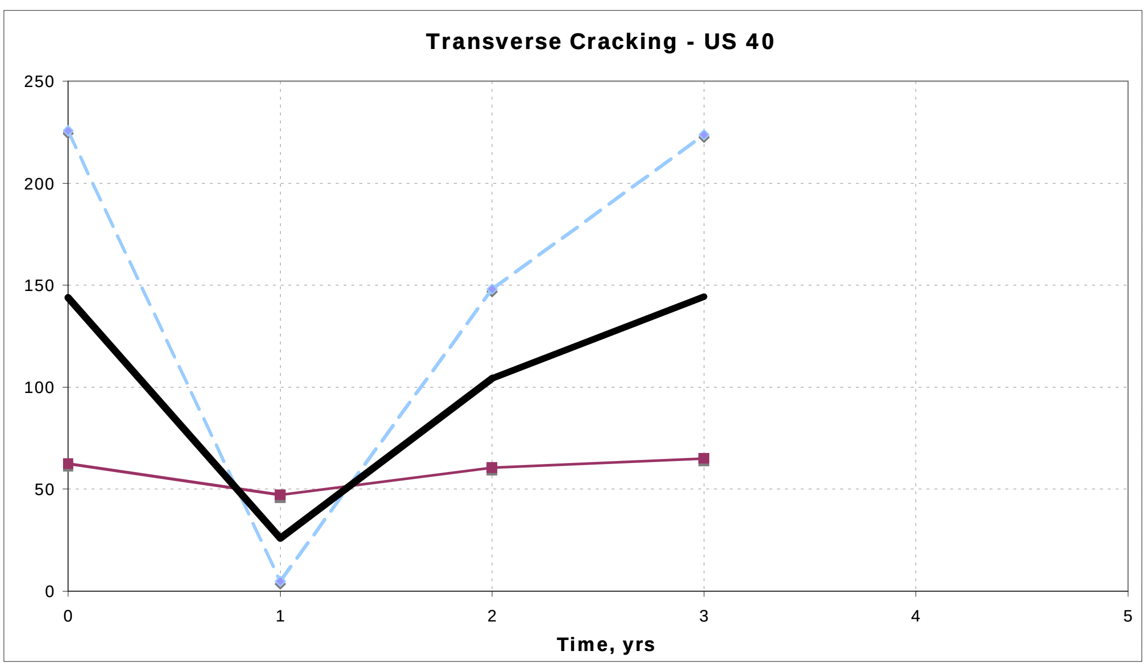


Figure 109. Transverse Cracking/500 ft Section for US 40, Golden

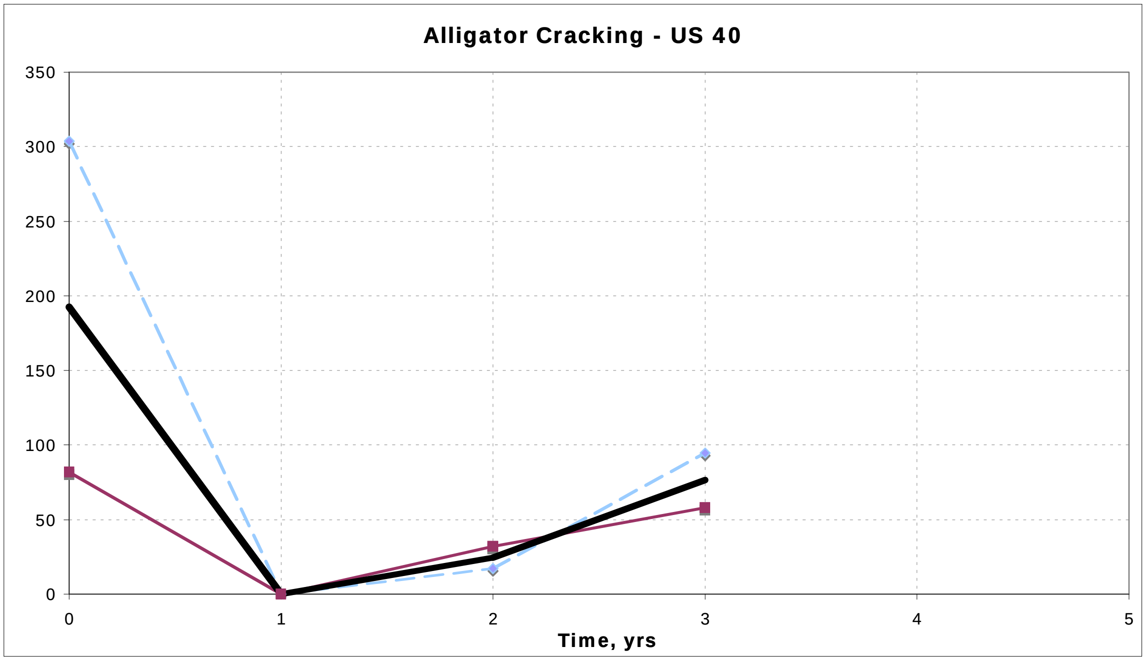


Figure 110. Alligator Cracking/500 ft Section for US 40, Golden

4.5 Ultra-thin Bonded Wearing Course

4.5.1 SH 58, Golden

Two evaluation sections were established on US 40, Golden in the driving lane and two sections were monitored in the passing lane. The results of three and four years of performance data on these sections and the average are shown in Figures 111 through 114. Two sections were covered with a new overlay after the third year.

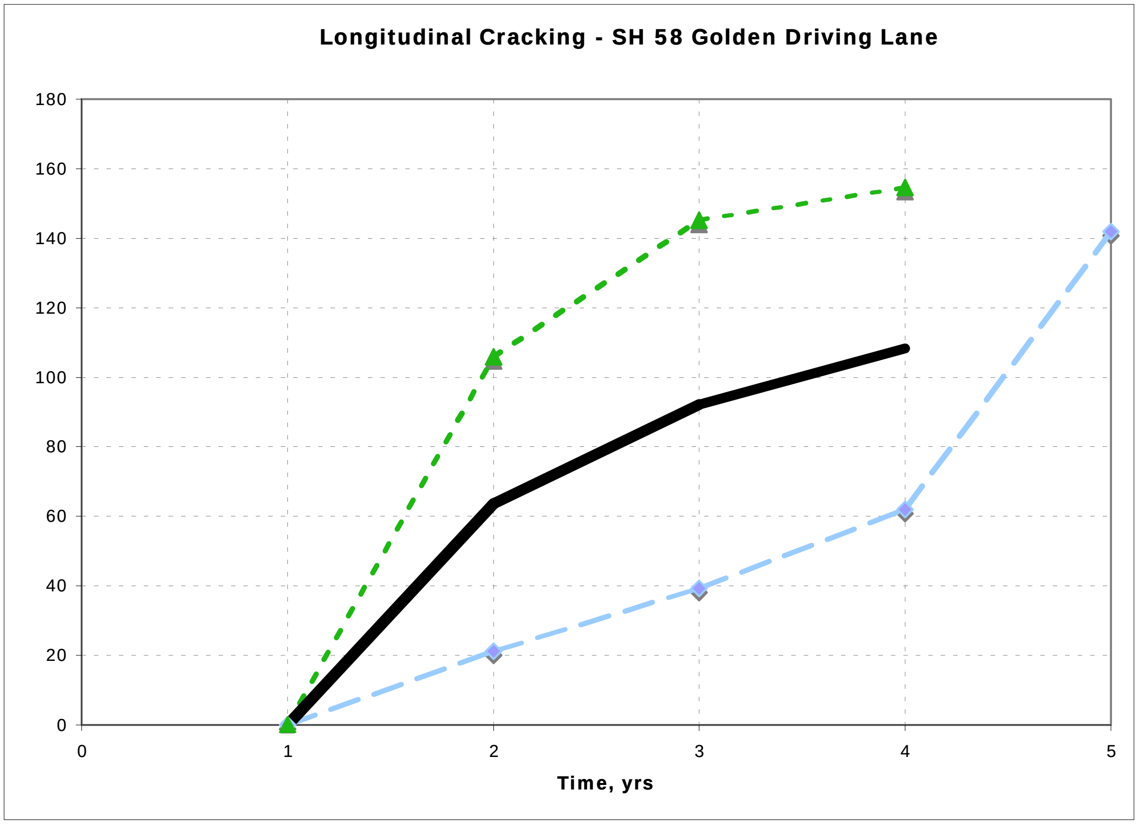


Figure 111. Longitudinal Cracking/500 ft Section for SH 58, Golden – Driving Lane

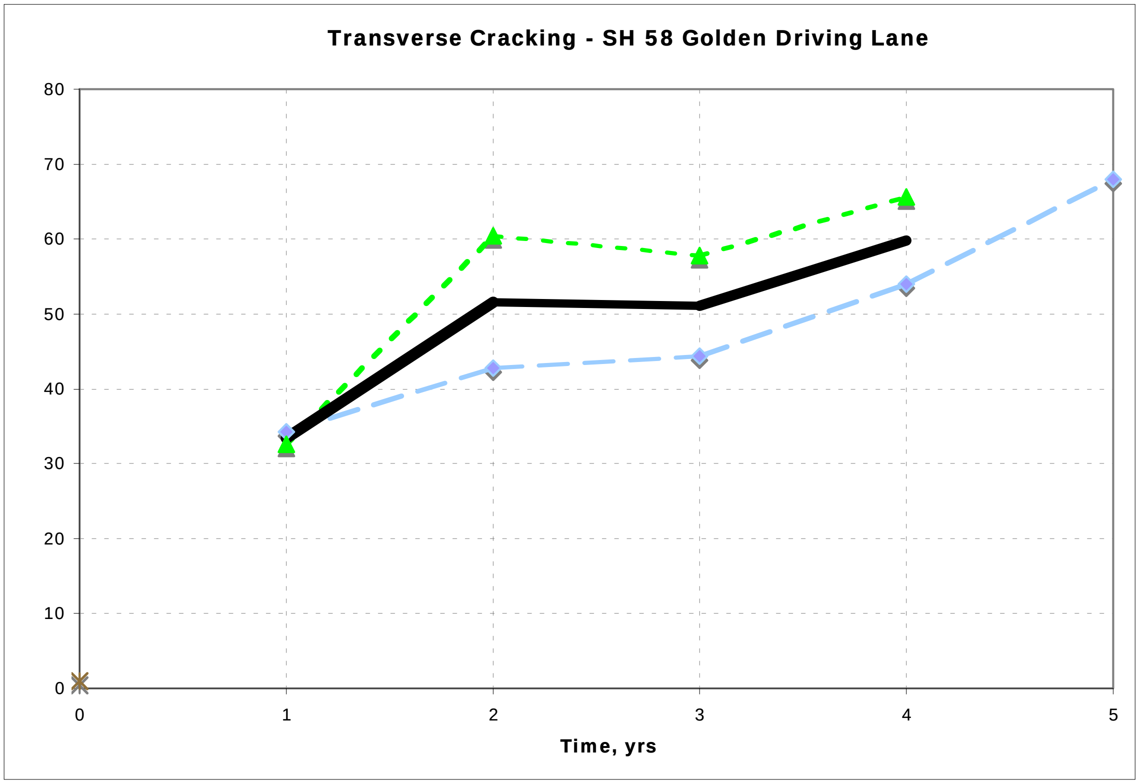


Figure 112. Transverse Cracking/500 ft Section for SH 58, Golden – Driving Lane

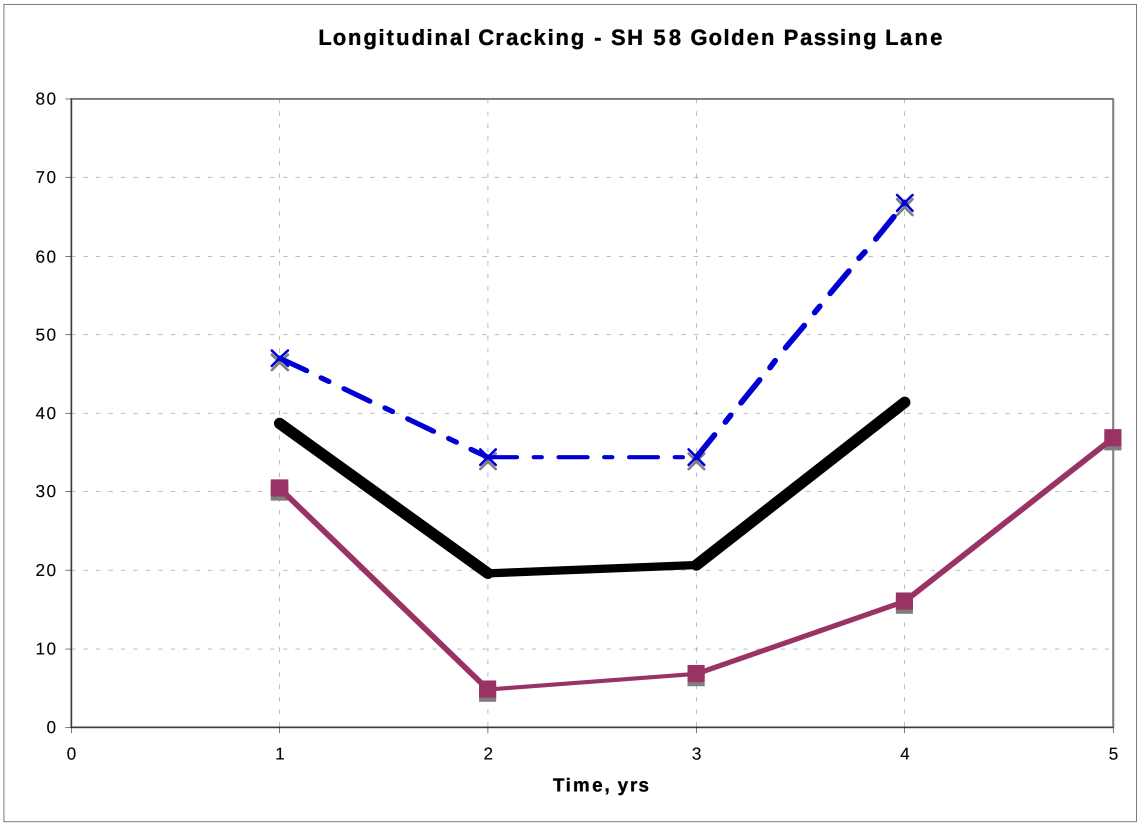


Figure 113. Longitudinal Cracking/500 ft Section for SH 58, Golden – Passing Lane

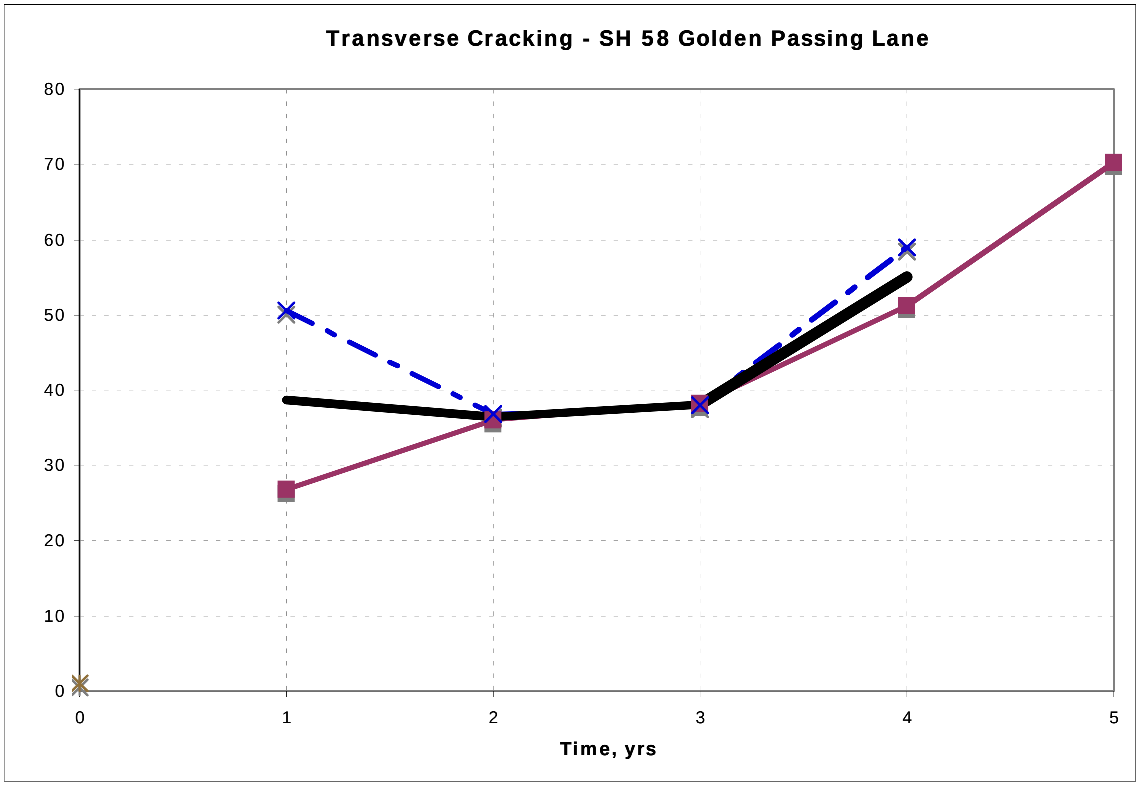


Figure 114. Transverse Cracking/500 ft Section for SH 58, Golden – Passing Lane

4.5.1 Table Mesa Drive, Boulder

Two evaluation sections were established on Table Mesa Drive in Boulder in the eastbound driving lanes one year after construction of the ultra thin bonded wearing course. Results of four years of performance data on these sections and the average are shown in Figures 115 and 116.

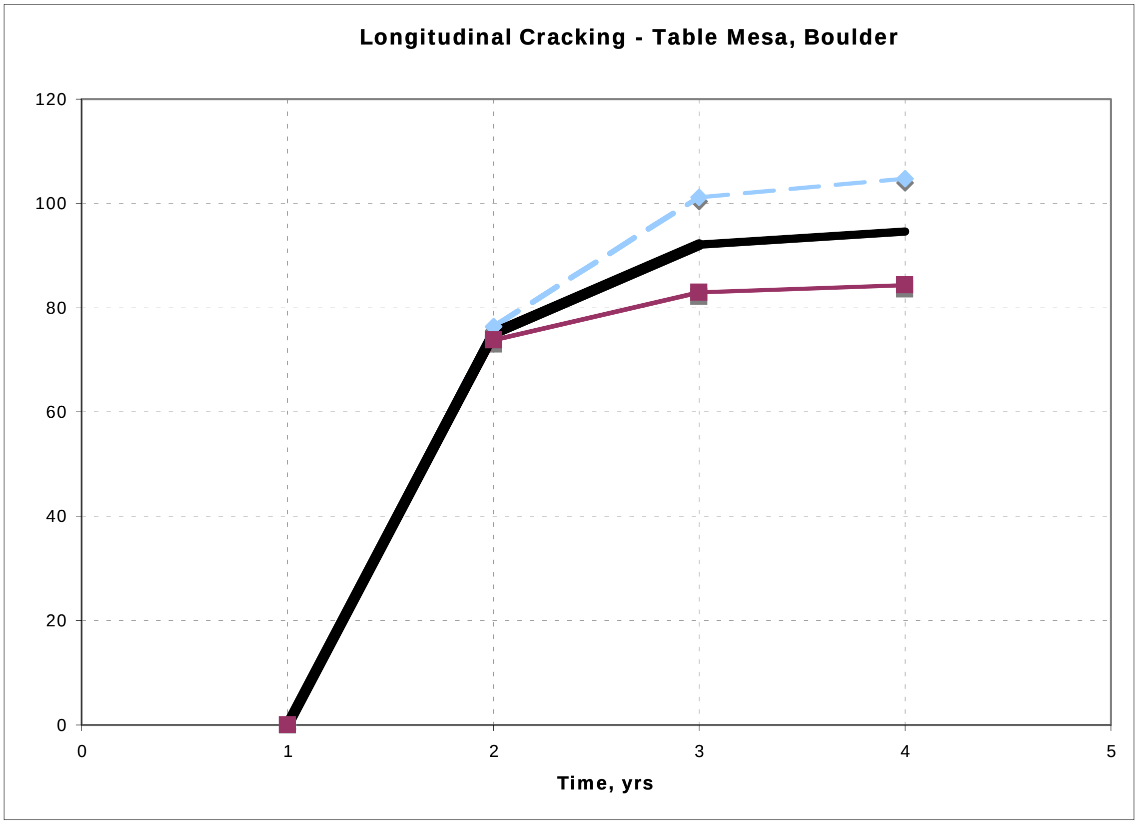


Figure 115. Longitudinal Cracking/500 ft Section for Table Mesa, Boulder

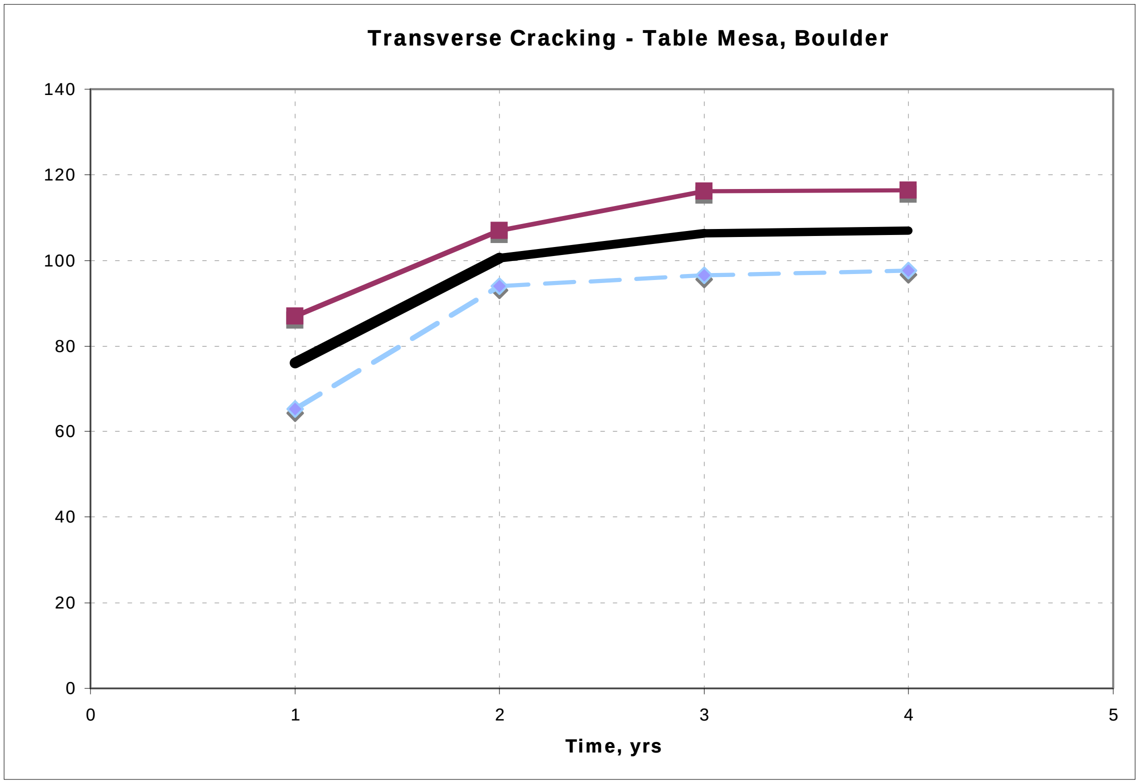


Figure 116. Transverse Cracking/500 ft Section for Table Mesa, Boulder

4.6 Concrete Pavement Microgrinding, I-70, Rifle

Three evaluation sections were established on I-70, Rifle in the eastbound driving lane prior to microgrinding operations. Two sections were ground and one section was left unground as a control. Four years of performance data on these sections and the average are shown in Figures 117 through 119.

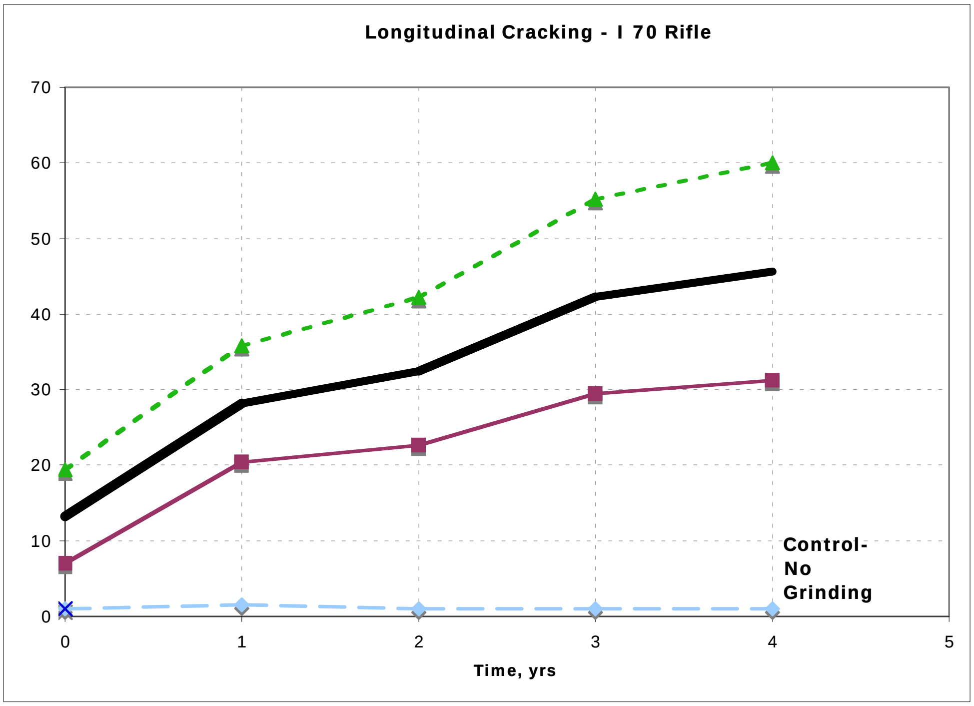


Figure 117. Longitudinal Cracking/500 ft Section for I-70, Rifle

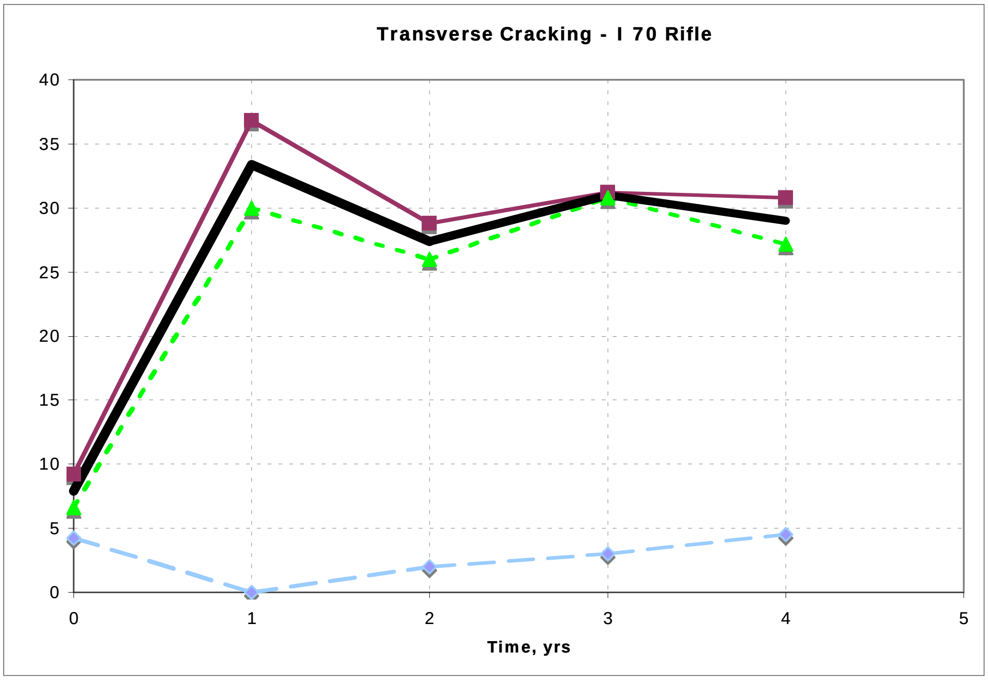


Figure 118. Transverse Cracking/500 ft Section for I-70, Rifle

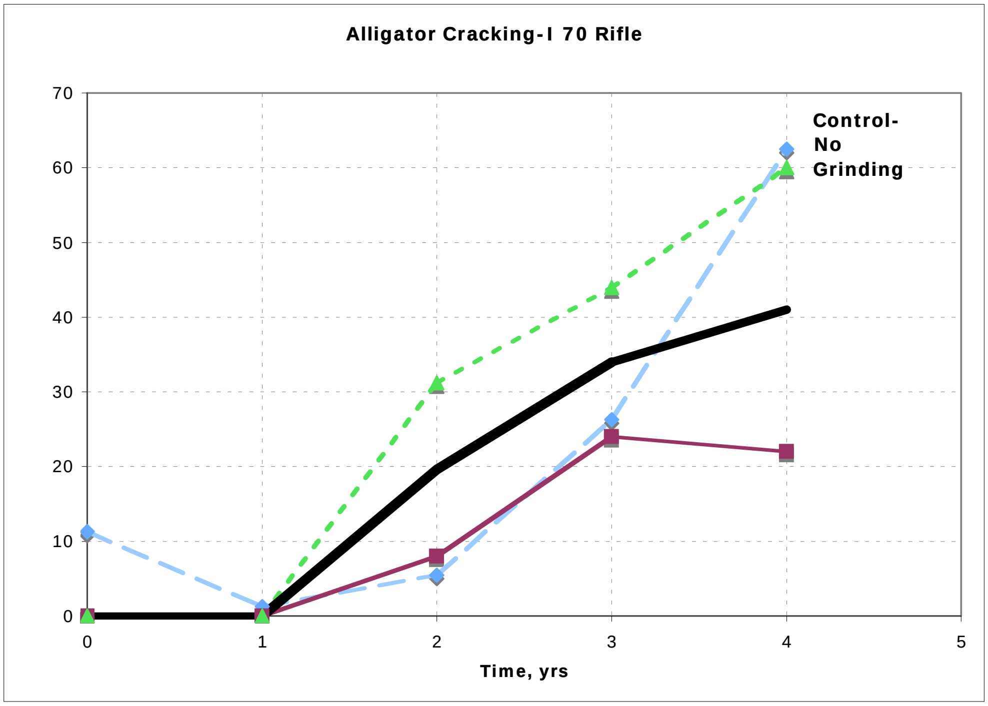


Figure 119. Alligator Cracking/500 ft Section for I-70, Rifle

4.7 Concrete Pavement Cross-stitching

Measurements were made of the width of the longitudinal crack where the deformed bars and fibreglass panels were inserted to prevent further crack separation in the pavement. No difference in crack width was measured in either the deformed bar sections or the fibreglass panel sections after four years. However, although the repaired crack has remained stable in the concrete panels that were repaired, a crack has propagated from the end of the crack in Section 8 (deformed bars) to the south for 15 feet. Also, a crack has propagated from the end of Section 9 (deformed bars) to the south for 60 feet.

4.8 Concrete Pavement Joint Resealing

The joints that had been resealed in 2006 remained sealed through 2007 and then began to fail in 2008 with approximately 5 percent of the joints separating from the sealant and approximately 10 percent of the sealant failing within the sealant. This failure increased in 2009 to approximately 30 percent of the sealant failing within the sealant revealing a void beneath as shown in Figure 120. This photo was taken immediately south of the southern test section at approximately MP 2.8. The control sections that had been established in 2006 had been sealed before the 2009 survey.

Figure 120. Sealant Failure on US 287, Campo

5.0 ANALYSIS

5.1 Crack Seal

The results of the analysis of variance (ANOVA) for the two test sites where two crack sealants were evaluated with and without exposure to $MgCl_2$ over time are shown in Tables 11 and 12. These results indicate a statistically significant difference at $\alpha < 0.05$ between the two products at both test sites. Also, the $MgCl_2$ has an effect on performance at the Estes Park test site for the first two years, but has no effect at the Lyons site. This result may indicate that elevation may play a role in sealant performance when $MgCl_2$ is present.

Table 11. Results of ANOVA for SH 7 Estes Park, Colorado

Effect on Performance Significant at α < 0.05				
Effect on Performance	**Time 1 Spring 05-Fall 06**	**Time 2 Fall 06-Spring 07**	**Time 3 Spring 07-Fall 07**	**Time 4 Fall 07-Spring 08**
$MgCl_2$	Yes	Yes	Yes	No
Product	Yes	Yes	Yes	Yes
$MgCl_2$*Crack Fill	No	No	No	No

'Signif = $\alpha < 0.05$

Table 12. Results of ANOVA for SH 66 Lyons, Colorado

Effect on Performance Significant at α < 0.05				
Effect on Performance	**Time 1 Spring 05-Fall 06**	**Time 2 Fall 06-Spring 07**	**Time 3 Spring 07-Fall 07**	**Time 4 Fall 07-Spring 08**
$MgCl_2$	No	No	No	No
Product	Yes	Yes	Yes	Yes
$MgCl_2$*Sealant	No	No	No	No

It is also interesting that the presence of $MgCl_2$ actually improved performance of the sealants at Estes Park.

It is also interesting to note that the best performing sealant was 90 percent effective after 1 year at Estes Park and after 4 years at Lyons. However, the best performing sealant was only 50 to 60 percent effective after 4 years at Estes Park. This may indicate that crack sealing frequency should be elevation dependent.

5.2 Thin SMA

Three thin SMA sections were evaluated on SH 74 Evergreen, SH 13 Rifle and I-70 Hanging Lake. Distress existing at Rifle and Hanging Lake was measured prior to construction of the SMAs. However, the SMA at Evergreen was built one year prior to distress evaluation. This makes it difficult to tell how effective the SMA at Evergreen has been at reducing distress. However, the longitudinal cracking appears to be reaching an asymptote after four years service. This is not true of the transverse cracking that has only reached an average of 2 feet in 500 feet of test section after four years.

The time it has taken distress to return to the original distress level at time zero (the time of the preservation treatment) will be used to evaluate the value of each preservation method. Table 13 below shows the approximate time observed for each pavement to return to the distress level existing at time zero.

Table 13. Analysis of SMA Performance

Test Section	Time to Original Condition, yrs	
	Longitudinal	Transverse
SH 13 Rifle	0	4
I-70 Hanging Lake WB	2.2	2
I-70 Hanging Lake EB	4	4+
Average	**2.1**	**3.3+**

5.3 Chip Seal

Six pavement test sections consisted of chip seals. These were US 34 Drake, SH 14 Briggsdale, US 285 Poncha Springs, US 285 Poncha Pass, SH 24 Leadville, and US 50 Swink. The average time to reach the original condition with respect to cracking for each treatment is shown in Table 14 below.

Table 14. Analysis of Chip Seal Performance

Test Section	Time to Original Condition, yrs	
	Longitudinal	Transverse
US 34 Drake EB	6*	0
WB	4	2.5
SH 14 Briggsdale ½”	n/a	2.5
3/8”	n/a	1.1
US 285 Poncha Springs		
No Fog Seal	3	2.5
Fog Seal	3.5	3.5
US 285 Poncha Pass		
No Fog Seal	4	3.5
Fog Seal	3	3.5
US 50 Swink EB DL	0	0
EB PL	0	0
WB DL	0	0
PL	2	0

* Extrapolated from 4 years

Although these pavements have begun to crack the surface condition of the pavement is in excellent condition with respect to texture, lack of chip loss, and lack of flushing. This indicates the asphalt and chip application rates are approximately correct for these pavements.

5.4 Thin Hot Mix Overlays

There were five thin hot mix asphalt overlay test section locations. These were US 6, Golden; SH 121, Littleton; US 550, Coal Bank Pass; US 550, Durango; and US 40, Golden. The average time to reach the original condition for each treatment is shown in Table 15 below.

Table 15. Analysis of Thin Hot Mix Overlay Performance

Test Section	Time to Original Condition, yrs			
	Longitudinal	Transverse	Alligator	Ravelling
US 6 Golden	1.5	0	2+*	
SH 121 Littleton	0	1+**		
US 550 Durango				
	Shoulders			
No HIPR 1-1/2”	3+	3+		
2”	3+	3+	3+	
HIPR 1-1/2”	3+	3+		
2”		3+		
	Driving Lanes			
No HIPR 1-1/2”	3+	3+	3+	3****
2”	3+	3+	3+	
HIPR 1-1/2”	3+	3+		
2”		3+		
	Passing Lanes			
No HIPR 1-1/2”	3+	3+	3+	
2”	3+	3+	3+	
HIPR 1-1/2”	3+	3+	3+	
2”	3+	3+	3+	
US 550 Coal Bank NB	1.5	4***	3+	3****
SB	1.5	4***	3+	
US 40 Golden	1.5	2.5	4***	

* A new overlay covered the test sections after the 2 year evaluation
** A new overlay covered the test sections after the 1 year evaluation
*** Extrapolated from 3 years
**** Raveling developed 3 years after treatment

5.5 Ultrathin Bonded Wearing Course

Two locations were selected for monitoring ultrathin bonded wearing course performance. These pavements were on SH 58, Golden and Table Mesa Drive in Boulder. Both of these projects were constructed one year before condition surveys were conducted so the original condition was unknown. However, the Table Mesa project was constructed over jointed concrete pavement with 12 foot joint spacing, so in each 100 foot evaluation segment there were 109 feet of transverse cracks. This means it took 3 years for the transverse joints to reflect to the surface and slightly longer for the longitudinal joints. Approximately 60 to 70 percent of the joints had reflected back after 1 year.

5.6 Concrete Pavement Microgrinding

Microgrinding on I-70 near Rifle does not appear to be effective at reducing cracking. The test sections that were ground continued to show increases in transverse and longitudinal cracking after the treatment while the control section that was not ground remained in approximately the same condition over time. The exception was with alligator or fatigue cracking where both test and control sections continued to crack over time.

5.7 Concrete Pavement Cross-stitching and Fiberglass Panels

Both the deformed bar cross-stitches and the fiberglass panels appear to maintain a connection between opposite sides of the crack they are installed to repair. No additional cracking surrounding the repair has occurred and the crack has remained at the same width for four years. However, the crack being repaired is continuing to propagate beyond the cross-stitch locations into the adjacent slab.

5.8 Concrete Pavement Joint Resealing

Joint resealing performed well for the first year after the resealing operation. However, the sealant began to fail after two years and at three years is separating from the joint and cracking within the sealant. Some joints have more than 30 percent failure. The cause is unknown but the sealant is demonstrating a brittle behavior not present during the first two years.

6.0 CONCLUSIONS

6.1 Crack Seal

Results indicate a difference between two crack fill products in regard to percent crack fill remaining over a four-year time period.

Exposure to $MgCl_2$ improved performance at Estes Park but not Lyons.

Performance of one of the sealants was significantly better at Lyons than Estes Park.

6.2 Thin SMA

The average time required for the thin SMA test sections to return to the original condition at the time of treatment was approximately 2.1 and 3.3 years, respectively for longitudinal and transverse cracking.

6.3 Chip Seals

The chip seals are all in good to excellent condition after three years with respect to texture, chip loss, and flushing. The time required for transverse cracking to return to the pre-chip sealed level ranges from less than 1 to 3.5 years. The time required for longitudinal cracking to return ranges from less than 1 to approximately 6 years. The effect of fog sealing over the chip seal was measured on US 285 Poncha Springs and Poncha Pass. The fog sealed chip seal is outperforming the non-fog sealed sections at Poncha Springs by about 0.5 years for longitudinal cracking and about 1 year for transverse cracking. However, the non-fogged section on Poncha Pass is outperforming the fogged section by 1 year for longitudinal cracking and is equal for transverse cracking.

6.4 Thin Hot Mix Overlays

The average time required for the 1-inch hot mix overlay test sections to return to the original condition at the time of treatment was approximately 1.5 years for both longitudinal and transverse cracking. The 1.5-inch overlays on US 550 north of Durango are performing better than this even in the sections not containing HIPR beneath. However, some transverse cracking has begun to appear on the shoulders in the sections with no HIPR, both for 1.5 and 2 inch overlays after three years.

6.5 Ultra-thin Bonded Wearing Course

Transverse joint reflection cracking occurred at approximately 60 to 70 percent after one year, and approximately 100 percent of all the UBWC test sections after three years.

6.6 Concrete Pavement Microgrinding

The treatment does not appear effective at reducing cracking and may be detrimental to performance.

6.7 Concrete Pavement Cross-stitching

The reinforcing bars and fiberglass panels are performing equally with no separation of the cracks within the repair. However, propagation of the crack into the adjacent slab has occurred. Future repairs should consider cross-stitching the adjacent slab before the crack propagates to measure the effectiveness of this technique at stopping the propagation.

6.8 Concrete Pavement Joint Resealing

The joint sealant is beginning to fail after only three years. Failure is occurring between the joint and the sealant and within the sealant. Sealant appears to have become brittle.

7.0 RECOMMENDED FUTURE RESEARCH

This research represents a good start to developing actual field performance for the preservation techniques used in Colorado. However, not all of the test sections established have begun to fail, so a clear understanding of the life-cycle of some of these treatments is lacking. For example, all of the chip seals placed in this research are performing well with respect to texture, chip loss, and flushing. This is an indication that the correct application rates for asphalt and chips were applied to these pavements. However, condition surveys should continue on these pavements to determine when the next chip seal should be applied so a measure of the chip seal performance can be judged. Also, the extensive test sections established on US 550 Durango could provide a much better understanding of the performance characteristics of hot in-place recycling if the condition of these test sections were monitored until differences in performance of the sections was observed.

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APPENDIX A

PREVENTIVE MAINTENANCE BEST PRACTICES MANUAL

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Introduction

The purpose of this manual is to provide a reference describing the best methods to use when conducting certain preventive maintenance procedures on asphalt and concrete pavements in Colorado. The methods described in this manual are based on a review of the literature, a series of interviews conducted in each Colorado DOT region, full-scale field test sections, and experience of the researchers.

For clarification, definitions have been offered (7) describing various types of pavement maintenance activities. These are:

- Preventive Maintenance
- Corrective Maintenance
- Emergency Maintenance

Preventive Maintenance: Activities intended to retard progressive pavement failures and reduce the need for corrective or emergency maintenance. Or, according to AASHTO: *"Preventive maintenance is the planned strategy of cost-effective treatments to an existing roadway system and its appurtenances that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system (without substantially increasing structural capacity)"*.

Corrective Maintenance: Performed after a deficiency occurs in the pavement, such as loss of friction, moderate to severe rutting, or extensive cracking. Sometimes referred to as "reactive" maintenance.

Emergency Maintenance: Performed during an emergency situation, such as a popout in concrete pavement or severe pothole that needs repair immediately. This also describes temporary treatments designed to hold the surface together until more permanent repairs can be performed.

Preventive maintenance is intended to prolong the interval before corrective and emergency maintenance are needed. And though all three types of maintenance are important, preventive maintenance activities should be the most cost-effective by prolonging pavement life.

Based on the interviews conducted in early 2005 three preventive maintenance processes are primarily used for asphalt pavements and three processes are primarily used for concrete pavements in Colorado. These processes are shown in Table 1.

Table A1 – Preventive Maintenance Processes Utilized in Colorado

Asphalt	Concrete
• Crack Filling (Sealing)	• Crack/Joint Sealing
• Chip Seals	• Cross-stitching
• Thin Overlays	• Diamond Grinding

These processes are the focus of this Best Practices Manual.

Crack Filling (Sealing)

There is a distinction made between crack filling and crack sealing in the literature (8) and also in the current Colorado DOT guidelines (23). Crack filling involves blowing a crack out with compressed air to remove debris and sealing the crack with an asphalt sealer. The cracks appropriate for crack filling are considered 'non-working' cracks, or cracks that do not move appreciably due to expansion and contraction or loading. Crack sealing is an operation that is applied to 'working' cracks defined by Galehouse (23) as "… *A crack in a pavement that undergoes significant deflection and thermal opening and closing movements greater than 2 mm (1/16 inch), typically oriented transversely to the pavement centerline*." Crack sealing consists of opening the crack by random crack saw or router to provide a definitive geometric cavity for the asphalt crack sealer to penetrate. Routing the crack prior to crack sealing is recommended by some agencies (6, 8) and considered cost effective. Canadian studies (13) describe what the geometry of the routed crack should be and the mechanism of the adhesion of the sealant to the routed crack face.

Routing cracks prior to filling with crack sealer is currently not practiced in Colorado. In fact, it has been advised against because of potential damage that could result (2) and the belief that it is uneconomical. However, recent observations in a neighboring state (6) shown in Figure 1 indicates that routing is being practiced prior to crack filling and may be economical when done under specific circumstances.

Figure A1 – Routing Cracks Prior to Crack Sealer Installation on I-25, NM

This manual will focus on crack filling as a preventive maintenance tool, since crack sealing is currently not practiced by CDOT.

The purpose of crack filling of asphalt pavements is to reduce the infiltration of water, anti-icing chemicals, and incompressibles into the pavement sub-structure. This reduces pavement degradation and helps extend service life (13). Crack sealing is most effective when applied to pavements in good condition (14), with low-to-moderate crack density, and where cracks show little or no branching as illustrated in Figure 2. Low to moderate density cracks are suitable for sealing, but high density crack patterns with excessive branching should be treated by other techniques.

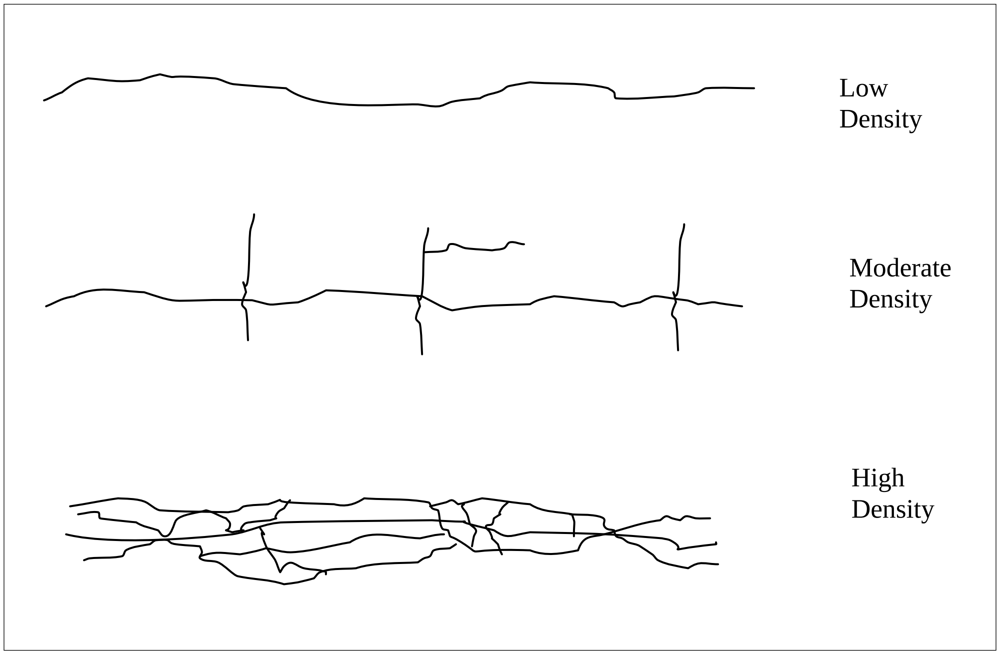


Figure A2- Example of Crack Density Levels

such as patching or resurfacing. Cracks with severe vertical distress such as cupping, faulting or show significant displacement when loaded are also unsuitable for crack sealing or filling and must be treated by patching or resurfacing after surface milling.

Pavement Selection

The condition of the pavement to be sealed has an effect on the performance of the crack filler. Only pavements that are structurally sound and show low levels of distress are candidates for crack filling. Table 2 has been adapted from the literature (8, 23) to give some guidance regarding the type of pavement that is a candidate for crack filling.

Table A2 - Pavement Candidates for Crack Filling

		Density		
		Low, Transverse @ 75-100 ft (23 – 30 m) Longitudinal @ CL	Mod, Transverse @ 50 – 75 ft (15 – 23 m) Longitudinal @ Lane CLs	High, Transverse @ 25 – 50 ft (8 – 15 m) Longitudinal @ Lane CLs+
Description/ Width, inch	Slight, <1/4	1	2	3
	Intermediate, ¼ - ½	4	5	6
	Severe, > 1/2	7	8	9

1, 2, 3, 4, 5	Crack filling is appropriate
7 ,8	Crack filling is appropriate or Crack filling after routing
6,9	Crack filling is not recommended

Timing

Crack filling should be accomplished as soon after cracks appear in the pavement as possible. In fact, the longer the cracks are left unsealed, the wider they become, and a less effective seal results. Therefore, on older pavements with wider and more numerous cracks, the process of crack filling becomes more of a corrective procedure and less of a preventive process. However, the methods used for narrow cracks is essentially the same as for wider cracks, although the interval of time between applications will probably be different. When cracks less than 1/8 inch were filled by approved methods in Vancouver, Canada, the effectiveness reported was between 7 to 9 years (15). Galehouse (23) indicates that cracks should be sealed at two to four years if the base is granular and from one to two years if the underlying pavement is concrete.

Hot poured polymer modified crack sealers do not penetrate very narrow cracks well. Therefore, the literature (8) indicates that cracks should be greater than 1/8 inch wide before hot poured asphalt sealers are used. However, when cracks first appear in asphalt pavements they tend to be very narrow, often less than 1/8 inch. So, how can cracks be filled when they first appear if the cracks are too narrow for hot poured sealants? One answer might be to use asphalt emulsions. However, emulsions have been reported to typically fail after one or two winters (16). Perhaps one reason they are not used by CDOT. Another may be to use hot poured sealants that are not polymer modified but meet the ASTM D1190 (5) standard for conventional asphalt cements. These sealants tend to be less viscous than polymer modified asphalts and should be capable of filling relatively narrow cracks if a small 1/8 inch nozzle is used. Another option is to used a combination of materials. Asphalt emulsion could be used to fill the crack, then an ‘overband’ of polymer modified asphalt on the pavement surface to ‘seal’ the emulsion and improve treatment life. This practice has been reported as very successful on low traffic pavements in milder Canadian provinces (17) with sealant durability of 7 to 9 years (18).

Hot poured sealants work best if the surfaces of the pavement within the crack are dry and free of magnesium chloride residue. Also, crack sealants should be placed six to twelve months or longer in advance of overlay construction. This means crack sealing should occur in the fall season. The fall is best because temperatures are cool, but not cold, making crack widths approximately average for the year; that is, wider than summer, but narrower than winter. This means the material is easier to get into the crack than it would be in the summer when the crack is narrowest, and an excess of sealer is not put in the crack as would happen in the winter months. Also, the pavement has a better chance of not having been sprayed with anti-icing chemical, yet, as it would in the spring.

Therefore, spring is the second best time to crack seal although some literature suggests spring the best time when emulsions are used (8). If moisture is suspected in the crack the moisture should be removed prior to sealing. This is best accomplished with a heat lance. The lance should be capable of 3000F with 3000 feet per second air velocity at the nozzle (2).

Cracks should be filled in advance of overlay construction. Hot mix asphalt causes crack sealer to liquefy. If the overlay thickness is two inches or less, the crack sealer can permeate through the overlay and cause a localized weakness in the pavement surface, slip plane, or bump. Therefore, crack sealer should be applied ahead of overlay operations as shown in Table A3.

Table A3 – Timing of Crack Fill Operations Prior to Overlays

Crack Width, in (mm)	Time Before Overlay, months
< 1/8 (3)	3
1/8 – ¼ (3-6)	6
¼ - ½ (6-12)	9
½-3/4 (12-19)	12

If conducted at the proper time, some report a life extension to the pavement of two to five years (8). Proper timing is considered to be a program where crack treatments are repeated more than once over the life of the pavement. This schedule is recommended after initial construction at an age of three to five years for the first application, then at an age of eight to ten years for the second application (8). The timing is dependent on the effectiveness of the sealant. Therefore, if the sealant opens up after five years, the next treatment must be applied to retain effective sealing of the pavement at that time. If the sealant separates in less than five years, the intervals between treatments will be shorter.

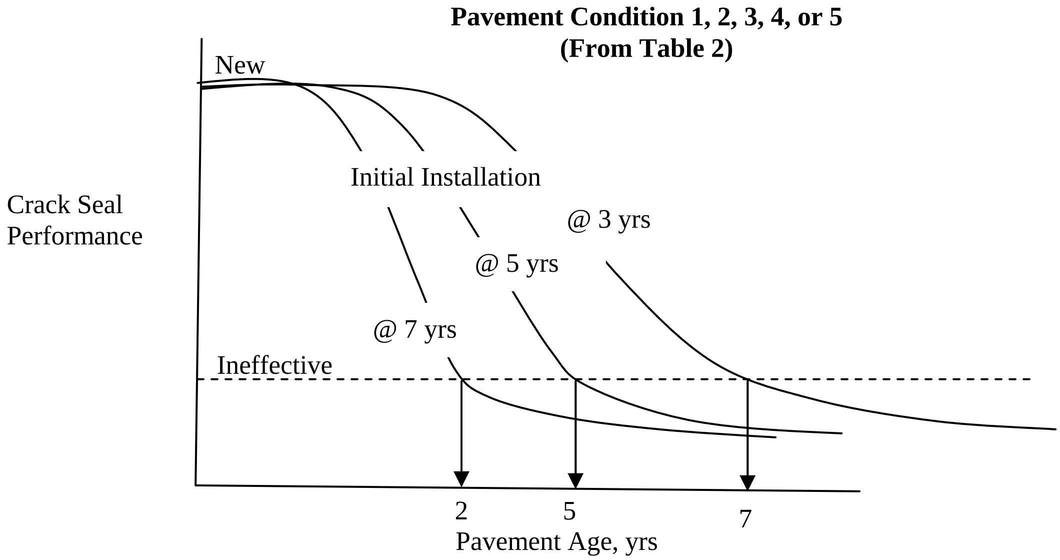


Figure A3 – Conceptual Relationship Between Initial Crack Fill Application and Performance

Conditions

The pavement should be as dry as possible before crack sealer is applied. As stated above, cool, but not cold ambient conditions should prevail. This means an air temperature of no less than 35F (8, 23) when sealing operations begin and an ambient temperature no greater than approximately 55F (8) during the sealing process. These conditions assure that the crack sealer will adhere to the pavement within the crack and that the crack will not be too wide, nor to narrow to accept the sealer.

If the crack contains moisture it must be dried using a heat lance to assure the best performance of the crack sealer. This process accomplishes two things: first, it dries the crack surfaces, and second, it softens the asphalt which provides improved adhesion. Reports in the literature indicate this process is essential if a minimum anticipated sealant life of five years is expected (11).

Some manufacturers claim their products will adhere as well in the presence of moisture as dry. However, unless these claims can be substantiated with quantitative evidence, it is best to avoid sealing operations under wet conditions.

Materials Selection

Crack filling with hot poured polymer modified asphalt cement is practiced in every region in Colorado as both a preventive and corrective maintenance procedure. Some regions have also used asphalt cement modified with ground tire rubber. Crack filling is usually conducted with CDOT maintenance personnel and is considered a routine practice.

The specification describing the material properties of polymer modified asphalt cement is ASTM D6690 (4). Currently, there is no ASTM or AASHTO standard describing ground tire rubber modified crack sealer.

Table A4 provides a summary of sealant performance in three Canadian provinces for products that met the appropriate ASTM specification at the time of the studies from 1995 to 2000. For comparison, the Denver metro area temperature range according to SHRP is 64C-22C at the 95% confidence level. Colorado currently uses the equivalent of an ASTM D3405 Type II.

Table A4 - Performance of Hot-Pour Sealants

	Vancouver	**Montréal**	**Ottawa**
Temp. range °C[a] (F)	–22 to 52 (-8 to 126)	–28 to 58 (-18 to 136)	–34 to 58 (-29 to 136)
Original sealant type	I	II	IV
1-year failure[b] level	0% to 5%	6% to 11%	7% to 55%
4-year failure level	20% to 23%	16% to 28%	not determined

[a] Pavement surface temperatures according to Superpave.

[b] Sum of debonding, splitting, and pull-out lengths.

From: Vancouver (17); Montréal (12); Ottawa (20)

An excerpt of ASTM D6690 is shown in Table A5 showing the properties for the three types of sealants above. ASTM D6690 classifies sealants as Type I to Type IV, and replaces ASTM D1190 (Type I) and ASTM D3405 (Type II).

Table A5 – Excerpt from ASTM D6690 (4)

	Type I	**Type II**	**Type IV**
Cone Penetration at 77F (25C), dmm	<90[a]	<90	90 to 150
Flow at 140F (60C)	≤5	≤3	≤3
Resilience, %	--	>60	> 60
Cyclic extension, %	50 at –18°C (5 cycles)	50 at –29°C (3 cycles)	200 at –29°C (3 cycles)

[a] 1dmm = 0.1 mm

Determination of which hot-poured sealants should be used under specific circumstances in Colorado should be evaluated over a 3 to 5 year period to evaluate performance of the sealants. Performance is reported to not be a linear relationship (12) and is represented as such conceptually in Figure 3.

Installation

Hot Pour Sealants

Crack treatment performance depends on three factors: 1) pavement condition, 2) product utilized, and 3) installation. Installation is affected by air temperature, pavement temperature and moisture. Installation includes crack cleaning, heating of the sealant, pouring, finishing with a squeegee, and protection with blotter materials, if necessary. Maximum crack opening occurs during the coldest months of the winter, as would be expected. Therefore, the most strain occurs to the sealant during this period when temperatures are low and extension is high, with movements of ¼ to 1 inch possible. This is the reason crack filling should be performed in the spring or fall. During these seasons temperatures are moderate and cracks are open to about average the annual cycle. However, the best conditions with respect to sealant adhesion to the crack face is in the summer when moisture is lowest and temperatures are highest. Unfortunately, if the sealant is applied in the summer, it will experience significantly higher strain in the winter and may fail due to excessive extension. Therefore, timing of the installation is a compromise placing the sealant at a time best suited to sealant adhesion (summer) when extension of the sealant could cause failure, and a time when moisture could cause poor sealant adhesion (spring or fall) but extension will be reduced.

The first step in crack filling is cleaning any debris from inside the crack. This is done using compressed air. Air pressure should not exceed 100 psi to assure damage does not result to crack faces. Figure 4 shows an operator cleaning a crack with compressed air.

Figure A4 – Cleaning Crack with Compressed Air

Moisture causes a lack of bonding of the crack filler to the crack faces. Therefore moisture must be removed prior to filling the crack with sealant. The compressed air will remove some moisture, but not all. Therefore, a heat lance as shown in Figure 5 should be used if additional moisture is present in the crack after the compressed air operation. The heat lance warms the crack surface and evaporates some of the moisture (21). The heat lance is not a cleaning tool and should only be used at temperatures below 950F and when the tip is 2 to 4 inches from the crack. The color of the hot end of the heat lance is a good indication of its temperature. If it is bright orange to bright red, the temperature is 1100F to 1900F; if it is dark red, 950F to 1100F; if it is black, 750F to 950F. Overheating of the crack leads to lower sealant adhesion (22). The heat lance is often most beneficial when crack sealing operations are done at air temperatures between 40 to 50F. However, at these temperatures in the morning, dew is often present, so more care must be applied in removing this moisture.

Once the crack is warmed using the heat lance it is ready to be filled with sealant. A pressure distributor as shown in Figure 6 is the recommended equipment for all crack sealing operations. A gravity feed pour pot is not recommended because of the difficulty associated with applying a uniform quantity of material.

After the crack is filled, all excess material must be scraped form the pavement surface using a squeegee as shown in Figure 7. Occasionally, traffic may pick up fresh crack sealer. If this occurs two solutions should be explored. One, evaluate the quantity of crack sealer being applied and make sure it is not excessive. The squeegee process should leave very little crack sealer remaining on the horizontal pavement surface. If the quantity is excessive, evaluate the squeegee operation, and make changes to reduce the quantity of crack sealer on the surface. If the quantity is not excessive, and traffic still picks up the crack sealer, apply sand as a blotter to the area on the horizontal pavement surface containing the crack sealer to reduce the adhesivity of the crack sealer. If this does not solve the issue, keep traffic off of the pavement until the crack sealer has cooled and does not stick to the tires.

Figure A5 – Heat Lance

Figure A6 – Pressure Distributor for Crack Sealing

Figure A7 – Squeegee Application After Crack Pouring

Preparing of Hot-Applied Sealants

Before being poured, crack sealant should be melted in a double-jacketed reservoir. Hot oil circulates in the jacket, preventing the direct heating of the sealant. This reduces sealant degradation. The melter is also equipped with a central agitator that must allow for efficient heat transfer throughout the sealant and for preventing hot spots. Gauges measure oil and sealant temperatures. The gauges must be calibrated every spring. It is highly recommended that supervisors and inspectors carry a hand-held thermometer to verify that the sealant gauge is indeed calibrated. An infrared thermometer can also be used to monitor temperature, but it becomes unreliable when the sealant emits fumes. Figure 8 shows one sealant melter that conforms to these requirements.

Figure A8 – Crack Seal Melter

Crack sealant degrades every time it is heated. Degradation is kept to a minimum by short heating times at temperatures below 350F. Reheating sealant must be avoided; a workday should begin with an empty crack seal melter. The overnight heating of sealant even at low temperatures, such as 175F, so the crew can begin work faster in the morning, must also be avoided.

Therefore, the basic steps involved in crack filling are described in the simplified diagram shown in Figure 9.

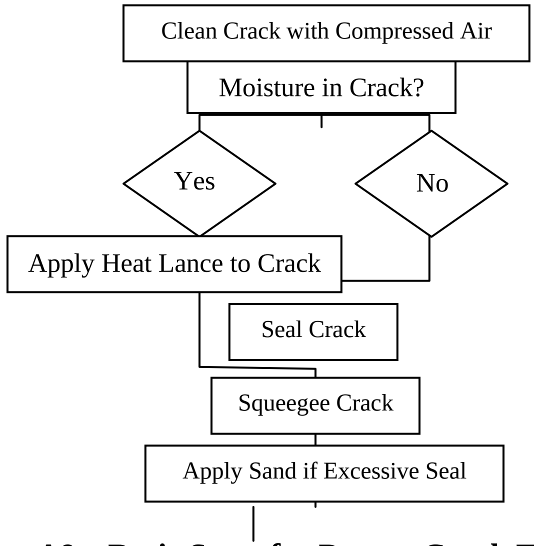


Figure A9 – Basic Steps for Proper Crack Filling

Cold Emulsion

Crack filling with asphalt emulsions should be done in late spring. Cracks that formed during winter are narrowest at this point and should not be moving. Since emulsion contains about 30 to 40% water, evaporation of this water can occur for the remainder of the spring and summer before winter exposure. Emulsions should be applied when air temperatures are above 65F, although they can be applied as cold as 50F. Complete curing takes eight to twelve hours. Low temperatures and a high relative humidity extend curing time. Freezing temperatures or rain will adversely affect emulsion performance and should not be expected within 24 hours of application. Therefore, conditions for emulsion application are best in late morning or afternoon. Very narrow cracks and cracks that are not moving can be treated in summer.

Cleaning Crack

Crack preparation is most important. A high percentage of material failure can be attributed to adhesion failure that results from dirty or moist cracks (11).

Dust and debris must be cleaned out of the crack or the crack sealer will adhere to the debris and not the crack face. As much debris as possible must be removed from the pavement surface so dust is not blown back into the crack just before it is sealed.

Debris and loose asphalt pavement fragments in and around the crack must be removed before sealing. This is best done with dry high-pressure air, free of oil. A compressor equipped with oil and moisture filters and providing at least 100 psi must be used. To check for oil or moisture, the compressor hose can be aimed at the side of a tire. Clean, dry air leaves no residue. Dry, high-pressure air removes some moisture from the crack.

Preparation and Application of Cold Emulsion

Asphalt emulsions used for crack filling are suspensions of asphalt, latex rubber, or other polymers in water. They are ready to use, but they can only be stored for a limited time. Emulsions where the water and asphalt have separated should not be used. Separation of the asphalt and water phases should only occur after the emulsions have been applied to the crack. The time required for this setting, or breaking, depends on temperature and humidity. Emulsions should become dry to the touch in 15 to 45 minutes. Complete hardening should take eight to twelve hours. Therefore, traffic should not be allowed on the sealed cracks until several hours after application of the sealer.

Cracks should be filled flush with the pavement surface with little or no excess.

Once the filler is poured, it should be left uncovered until fully cured.

Chip Seals

Chip seals in Colorado are primarily constructed using crushed natural mineral aggregates and either medium setting or rapid setting anionic or cationic emulsified asphalts. This type of chip seal is the basis for the following discussion.

Pavement Selection

Chip seals have two purposes: 1) they are intended to seal the surface of an asphalt pavement from moisture and air, and 2) they can improve the frictional characteristics of a pavement. Application of chip seals should be done when an asphalt pavement has just begun to oxidize and change color to a faded gray. Chip seals should not be applied to pavements with distress such as high severity cracking, raveling, or potholes (24) and application of a chip seal to rutted pavements should be evaluated in advance to determine if another preventive maintenance treatment would be more appropriate. The Long-Term Pavement Performance (LTPP) program of the Strategic Highway Research Program (SHRP) included a study focusing on the timing of pavement maintenance actions. It found that roads containing high levels of distress when chip seals were applied had a probability of failure that was two to four times greater than roads in good condition when the chip seals were applied. It also found that the chip seals tended to provide better economics with respect to preventing future distress better than the other treatments evaluated (25). Survey respondents in a recent NCHRP Synthesis (26) indicated that determining when to use a chip seal could result from a combination of factors, ranging from formula-driven algorithms to birthday sealing or visual evaluation of the pavement surface.

Timing

Personnel from nine states in the NCHRP Synthesis (26) indicated they got excellent service life from chip seals. These personnel included maintenance forces from Colorado DOT in Alamosa, Grand Junction, Montrose, Sterling, and Trinidad. These groups indicated they use chip seals as a preventive maintenance tool on a five year cycle. These agencies reported an expectation of six year service life from chip seals on this cycle. This is important because the construction cycle is shorter than the expected life cycle of the seal, which provides an extension to the service life of the pavement, in other words, preventive maintenance. However, interviews (27) with maintenance personnel in all of the maintenance sections in Colorado indicated that some chip seals are still applied to pavements in poor to very poor condition using a ‘worse first’ policy.

Conditions

Weather conditions are often responsible for premature failure of chip seals (28). Because the performance of anionic emulsions depends on evaporation for developing adhesive properties, ambient and pavement temperatures, relative humidity, wind velocity, and precipitation all affect early performance of chip seals constructed with this type of emulsion. In addition, cationic emulsions are also susceptible to early failure if moisture in the form of precipitation contacts the chip seal before breaking or setting of the emulsion occurs. Ideal chip seal weather conditions are those with low relative humidity, low wind velocity, and increasing temperatures during the day the chip seal is constructed (29).

Ambient Temperature

Ambient air temperature affects the performance of chip seals (28). Warm, but not hot, ambient air temperatures help reduce emulsion set time and promote adhesion between the emulsion residue and the aggregate chip and between the aggregate chip and the pavement surface. Specifications from several states (30, 31, 32, 33) require ambient air temperature to be a minimum of 50F (10C) when using emulsions for chip seals. However, according to a recent NCHRP Synthesis (26) the Indiana DOT allows placement in air temperatures below 50F if the aggregate has been heated to a temperature between 120F to 150F (34). High temperature can adversely affect emulsion set time, also. Consequently, California Department of Transportation specifies a maximum ambient air temperature of 110F (43C) for chip seal construction (35).

Pavement Temperature

The temperature of the pavement to be sealed affects binder adhesion to the aggregate chips and pavement surface. If the surface temperature is too low, poor adhesion can result because of slow setting of the emulsion so the Asphalt Institute recommends a surface temperature of 70F (21C) when constructing chip seals (36). However, experience by the author indicates that when air temperature is predicted to increase during the day chip seal construction can begin at pavement surface temperatures below 70F. However, high pavement temperatures can also be a problem. If the viscosity of the emulsion residue is too low, aggregate chips are not secured to the pavement surface with enough adhesion and can be picked up by traffic or pneumatic rollers jeopardizing the chip seal. Michigan DOT (37) limits chip seal construction to surface temperatures less than 130F (54C) and Ohio DOT (38) limits surface temperature to 140F (60C).

Precipitation

Chip seals should never be constructed if precipitation is expected before the emulsion has time to set. Emulsified asphalt is a mixture of asphalt and water and soluble in water. Therefore, if rain occurs before the emulsion sets, it is possible the rain will wash the emulsion off the pavement surface requiring the chip seal be reconstructed. However, in the event that rainfall occurs before the emulsion is set, it may be possible to save the chip seal using the following steps: 1) cover the emulsion as soon as possible with at least two times the design quantity of aggregate, 2) make one pass of the surface with pneumatic rollers to just set the chips in place, 3) do not allow traffic on the surface until the emulsion has set.

Wind

Wind decreases the set time for asphalt emulsions. Therefore, the wind speed has an effect on how close the rollers should be to the asphalt distributor during construction of the chip seal. The higher the wind speed, the faster the set, and the closer the rollers should be to the distributor. In addition, higher wind speed allows for earlier sweeping and removal of traffic control. However, if wind speed is too high, e.g. 25 mph or greater, the spray pattern of the asphalt distributor could be affected. In this case, either a shield should be installed to deflect the wind, or construction operations should cease until wind speed decreases. Also, wind can blow emulsion across to the adjacent traffic lane, creating potential claims from passing motorists.

Materials

Aggregate Chips

The aggregate used for chip seals defines how well the seal will perform. The best aggregates have high durability, abrasion resistance, contain little, if any dust, and are as nearly one-sized as possible. The surface texture should be rough and the aggregate should be resistant to polishing under traffic. Some believe aggregates carry an electrostatic charge. It seems reasonable that calcareous aggregates such as limestone or dolomite could be positively charged while silaceous aggregates like granite would have a negative charge. This would mean that anionic emulsions being negatively charged should adhere better to calcareous aggregates and cationic emulsions should adhere better to silaceous aggregates. The authors found little in the literature (46) that supports or contradicts this notion, although there is some evidence from interviews that does support this theory (27). More work should be done on this subject since many chip seals are constructed with aggregates produced from sand and gravel sources which often contain both silaceous and calcareous rocks.

Gradation

Aggregate gradation has much to do with how well a chip seal will perform. Although graded aggregate seals have been successfully constructed (43), most agree that the closer to one-sized a chip seal aggregate is, the higher the probability of success. The reason for this is that if the chip seal aggregate contains a wide range of aggregate sizes the smaller sizes are likely to become embedded in the emulsion before the large sizes. If this happens, the large sizes will not have adequate binder to hold them in place and may become dislodged and become potential projectiles. One-size aggregates produce a more uniform thickness and consequently a more consistent embedment in the asphalt binder. This contributes to improved aggregate retention, friction, and drainage characteristics (40).

Larger aggregates such as ½-inch, or even ¾-inch can be used in chip seals. The advantages are increased asphalt binder and therefore, more sealing potential. Also, since these aggregates are larger, they have a wider margin for error with respect to asphalt quantity. However, the disadvantages of large sized chip seal aggregates include increased tire noise, and increased cost due to higher binder volume. Increased risk of windshield damage has been offered as a disadvantage, as well. Although, this may be true, the adherence of large sized stones should be equal to smaller aggregates if the design binder quantity and design chip quantity are appropriate. Also, larger stones have more mass than smaller stones. Therefore, more energy would be required to dislodge them and make them projectiles.

The most important factor regarding gradation is the amount of material finer than the No. 200 screen. Colorado (30) limits this to 1%. Other states have similar requirements (30). Material passing the No. 200 screen can prevent asphalt binders from adhering to the surface of the aggregate resulting in retention problems (43).

Shape

Aggregate shape is important to the success of a chip seal. An angular, blocky shape is preferable to a flat and elongated shape. Flat and elongated shapes tend to become submerged in the asphalt binder resulting in a flushed surface. Cubical and angular shapes do not tend to become reoriented under traffic (39), so flushing is much less likely. Cubical and angular shapes also provide a more predictable shape for determining asphalt quantity during the design and construction phase of the project and cubical and angular shapes interlock better than flat and elongated shapes providing better long-term particle retention and stability. Flat and elongated particles can be determined by laboratory testing using either the flat and elongated test methods for Superpave or the Flakiness Index (41, 42, 43).

Aggregate for chip seals should be fractured mechanically. Rounded aggregates displace easier and do not interlock well. Therefore, Colorado requires that 90 percent of the plus No. 4 sizes have at least two faces fractured by mechanical means when tested using Colorado Procedure 45 (30).

Moisture

A damp aggregate provides a better surface for asphalt emulsion to adhere to. Therefore, aggregate stockpiles should be sprayed with water one to two days before the start of the chip seal operations. This spraying accomplishes two things: 1) the moisture provides a mechanism for the emulsion to absorb into the voids of the aggregate by capillary action, and 2) the spraying may wash off some minus No. 200 particles, reducing the chance for this dust to interfere with the adhesion of the binder to the aggregate surface.

Toughness and Soundness

Chip seal aggregates must be very tough, sound particles. The Los Angeles Abrasion test (45) is specified by most agencies to qualify aggregates for use as chip seal aggregates. Colorado DOT specifies a maximum of 35 percent loss (30). However, some studies (47) have shown that for high traffic pavements in excess of 7500 vehicles per day per lane, 35 percent loss may be too high and should be reduced to no more than 25 percent loss.

Emulsified Asphalt

Emulsified asphalt for chip sealing should have a consistency that allows for uniformly covering the pavement surface while not so fluid that it forms puddles or flows across the pavement. The binder should develop adhesion quickly upon application of the cover aggregate chips.

Two types of emulsified asphalt are specified by Colorado DOT (30): 1) Cationic CRS-2P and 2) Anionic HFRS-2P. Please note that in Table 702-4, the 'max' and 'min' columns are reversed.

Construction

Five types of equipment are needed to construct a chip seal. These are:

1. Asphalt Distributor
2. Aggregate Chip Spreader
3. Rollers
4. Dump Trucks
5. Brooms

Asphalt Distributor

The asphalt distributor is a self-propelled vehicle with a tank for holding the asphalt emulsion and a spraybar for applying the emulsion to the roadway. Although it is not specifically required by CDOT in Section 409.05 (30), computerized distributors which control the application rate of the emulsion are highly desirable. However, even with computer control, it is recommended that each nozzle of the distributor be calibrated prior to use. Research has shown that even when new, nozzle output can be highly variable (47). Also, before spraying operations begin the angle of each nozzle in the spraybar should be checked. The angle of each nozzle must be the same and in accordance with the manufacturer's recommendation. Angles of from 15 to 30 degrees from horizontal are typical as shown in Figure 10.

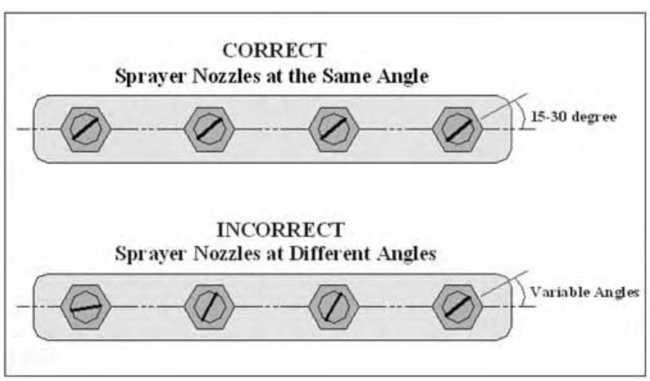


Figure A10 - Nozzle Alignment.

Aggregate Chip Spreader

The aggregate chip spreader must apply a uniform, even layer of aggregate across the full width of the binder. Figure 11 shows an example of a typical self-propelled aggregate spreader.

Figure A11 – Self-Propelled Aggregate Chip Spreader

A self-propelled spreader, equipped with a receiving hopper in the rear, belt conveyors to carry the aggregate to the spreading hopper, and a spreading hopper with adjustable discharge gates, is the preferred equipment for use. A discharge roller that assists in ensuring uniform transverse application rates is often located at the bottom of the discharge gate. Some equipment is available with variable-width spreading hoppers that hydraulically extend to adjust to changing spread widths. Many chip spreaders are equipped with computerized controls that allow the spread rate to remain constant as the speed of the spreader changes. This ensures a constant application rate, regardless of travel speed. Also, spreaders should be equipped so larger aggregates are forced to hit the emulsion before smaller aggregates. This is in accordance with CDOT specifications Section 409.05 (30).

The time required between emulsion application and chip application varies. If the chips are allowed onto the emulsion too soon, the chips may roll over because the emulsion has not had time to develop sufficient viscosity. And, if chips are kept off the surface too long, the emulsion may partially break, reducing adhesive ability. Therefore, the time allowed before chips are allowed onto the emulsion is critical to a successful chip seal.

One method the author recommends based on observations of experienced chip seal contractors consists of casting some chips onto the emulsion coating the pavement surface at varying time intervals after the emulsion has been sprayed. If the chips roll over in the emulsion, not enough time has elapsed. If the chips stick to the surface and do not roll over, it is time to apply the aggregate chips.

Rollers

Rollers follow the aggregate spreader to force the aggregate into the asphalt emulsion. This provides initial embedment of the aggregate into the emulsion and reduces the chance that aggregate will become dislodged after opening to traffic. This operation has been well documented and has changed little since early evaluations (46). The distance between the rollers and the chip spreader should be adjusted so the rollers do not pick up excess chips. This must be evaluated in the field on each project since the adhesion of the chips to the emulsion will vary with substrate temperature, wind speed, emulsion properties, moisture in the chips and roller speed, to name several. Two types of rollers are used for chip seals in North America, 1) rubber tire (pneumatic) and 2) steel-wheel. There is some controversy regarding use of steel-wheel rollers on chip seals.

<u>Rubber Tire Rollers</u>

Rubber tire, or pneumatic, rollers are used on virtually every chip seal project. The number of rollers may vary, but there should always be at least two of these rollers. Rubber tire rollers work well on chip seals because the contact pressure between the roll and the aggregate chip will not exceed the tire pressure of the roller. This pressure may vary but should be a maximum of approximately 80 to 90 psi.

Roller speed is important. If the rollers are moving too fast, chips may become dislodged during rolling, jeopardizing performance of the chip seal. Speed should be no faster than a fast walk, or about 3 miles per hour.

<u>Steel Wheel Rollers</u>

Use of steel-wheeled rollers is controversial. Some believe a steel-wheel roller provides a smoother surface than the rubber tire roller and should always be used following rubber tire rolling. Others believe use of the steel wheel roller is risky because of possible crushing that can occur to the aggregates under the very high stresses imposed by such rollers. In addition, unless aggregates are of very uniform size, the larger aggregates will support the load of the rigid steel drum, preventing any contact with smaller aggregates. And, steel wheel rollers may not contact aggregates in rutted areas of the pavement leaving these aggregates unseated.

Vacuums and Brooms

The surface of the pavement requires cleaning before the chip seal is applied and the chip seal requires cleaning of excess chips before traffic is allowed on the new surface. Two different types of equipment are used for these purposes: 1) vacuums, and 2) brooms.

Vacuums

Vacuums work by removing dust, debris, loose chips and moisture from the pavement surface through brooms and vacuum or just vacuum alone. A vacuum sweeper consists of brooms and a vacuum system. The brooms sweep debris or moisture to a centrally located vacuum system which lifts the materials and deposits them into a storage tank. A vacuum pickup removes dust, debris, loose chips and moisture by vacuum, only. The advantage of the vacuum pickup is that it does not contact the surface of the chip seal with brooms and therefore, causes less potential damage than brooms or sweepers.

Brooms

Rotary push brooms can be used to clean the pavement surface prior to construction and also remove excess chips from the pavement surface. When used to remove excess chips rotary brooms must be used with extreme caution because too much downward pressure on the broom can destroy the fresh chip seal. Therefore, the skill of the broom operator is important to the success of the chip seal and the amount of time that elapses between chip application and brooming is a function of operator skill.

Fog Seal

A fog seal may be applied to the chip seal surface following brooming and vacuum operations and before striping. This is an optional technique consisting of a light application (less than 0.10 gallons per square yard) of diluted asphalt emulsion (CSS-1h or SS-1h) sprayed on the chip seal surface prior to striping. The fog seal provides two potential benefits: 1) it makes the pavement surface dark, emphasizing the new striping, and providing improved visibility, and 2) it provides a small amount of extra binder to aid in chip retention. Other than these two potential benefits, no economic benefit has been reported.

Thin Overlays

Thin overlays for preventive maintenance are defined as hot mixed asphalt concrete pavement (HMA) overlays applied to existing pavements for the purpose of restoring surface texture or removing permanent deformation. Thin overlays are used for pavements where chip seals are considered inappropriate. Thin overlays are HMA of 1.5 inches compacted thickness or less. This includes, but is not limited to, dense graded HMA, stone mastic (matrix) asphalt (SMA), and ultrathin bonded wearing courses.

Pavement Selection

The asphalt pavement to be restored using thin overlays should be in good to fair condition. This means cracking should be of low to moderate severity and should have been crack sealed between 6 and 12 months of the thin overlay application. Raveling should be of low to moderate severity with depressions caused by stripping of the surface no greater than ¼-inch in depth. There should be no potholes. However, if potholes have been adequately repaired by cutting out the affected area and placing and compacting new HMA, thin overlays may still be effective.

Timing

Thin overlays must be constructed during the warmest part of the construction season. Temperatures of the surface and ambient air must be in accordance with Section 401 of the standard specifications (30) and not less than 60F. This is because compaction of overlays of less than 1.5 inches is very difficult under the best conditions since temperature loss of thin asphalt mixtures occurs very rapidly. In fact, the time required for a 1 inch lift to cool to a temperature (175F) where compaction is very difficult is 6 minutes if the air temperature is 60F and the mixture is delivered to the paver at 300F (49).

Conditions

Weather conditions are critical to the successful construction of thin overlays. Compaction is difficult under the best conditions for dense graded HBP when applied in thin lifts as described in the previous paragraph, so weather must be warm and dry before attempting this type of construction. Although SMA and thin-bonded overlays do not require the level of compaction as dense graded mixtures, they do require rolling to seat the aggregates in place and cool, or wet weather is detrimental to this objective.

Materials Selection

Hot Mixed Asphalt

Hot mixed asphalt (HMA) used for thin overlays must meet the requirements of Section 702.01 for asphalt cements and Section 703.04 for mineral aggregates.

SMA
SMA used as a preventive maintenance treatment must meet requirements specified in CDOT Special Provisions.

Ultrathin Bonded Wearing Course
Ultrathin Bonded Wearing Courses used as a preventive maintenance treatment must meet requirements specified in CDOT Special Provisions for these products.

Installation
Construction of thin HBP and SMA should follow procedures specified in Sections 401 for mixing, hauling, laydown and compaction (30).

Joint Resealing

Joint resealing consists of replacing the joint sealer in joints or cracks of Portland cement concrete (PCC) pavements. The objective of resealing joints in concrete pavements is to return sealant integrity to the joint to prevent further intrusion of moisture or incompressible solids into the joint. Reducing moisture infiltration into the joint reduces the potential for pumping and consequent loss of subgrade strength, and eliminating entry of incompressibles into the joint reduces the potential for joint damage caused by compressive forces.

Pavement Selection

When to reseal joints in concrete pavements is an important decision. If done too early in the life of the joint seal, funds may be wasted, and if done too late, deterioration may have begun reducing the effectiveness of the sealant. Some agencies replace joint and crack sealant when some percentage of the existing joint or crack sealant has failed. This varies between 25 and 50 percent according to Evans, et al (50). They go on to recommend a more analytical method for determining the best candidate pavements for resealing in their updated version of Strategic Highway Research Program (SHRP) report H-349 (50). The method they recommend uses a worksheet shown in Figure A12 to estimate 1) sealant condition, 2) pavement condition, 3) traffic, and 4) climate. The decision whether to reseal is then determined from Table A6. This system results in a seal condition number (SCN) which is a function of the number of low, medium and high-severity seal conditions which are a function of seal leakage and stone intrusion of the seal. Pavement condition is evaluated based on the presence of pumping, faulting, D-cracking, compression spalling at the joints, and blowups. Environment is evaluated based on the potential for moisture intrusion and freeze-thaw using the criteria shown in Table A7. Traffic is evaluated based on three levels of traffic volume as shown in Table A8.

Seal Condition			
	Low	Med	High
Water entering, % length	< 10	10-30	> 30
Stone intrusion	L	M	H
Seal Rating	Good	Fair	Poor

Environmental Conditions	
Avg annual precip., mm	
Days ≤ 0°C	
Avg low / high temp, °C	
Climatic Region [a]	WF WNF DF DNF

Traffic Conditions			
ADT (vpd); % Trucks			
Traffic Level [b]	Low	Med	High

Pavement Condition			
	Low	Med	High
Expected Pavement Life, yrs.	> 10	5-10	< 5
Average faulting, mm	<1.5	1.5-3.0	>3.0
Corner breaks, % slabs	< 1	1-5	> 5
Pumping, % joints	< 1	1-5	> 5
Spalls >25 mm, % slabs	< 5	5-10	>10
Pavement Rating	Good	Fair	Poor

Current Joint Design	
Sealant age, yrs	
Avg. sealant depth, mm	
Avg. joint width, mm	
Avg. joint depth, mm	
Max. joint spacing, m	

[a] See table 2.
[b] See table 3.

Figure A12 – Concrete Pavement Joint/Survey Form (50)

Table A6 – Decision Table for Resealing Concrete Joints

Sealant Rating[a]	Pvmt. Rating	Traffic Rating	Climatic Region: Freeze – Wet	Climatic Region: Freeze – Dry	Climatic Region: Nonfreeze – Wet	Climatic Region: Nonfreeze – Dry
Fair	Good	Low	Possibly	Possibly	Possibly	Possibly
Fair	Good	Med	**Yes**	Possibly	Possibly	Possibly
Fair	Good	High	**Yes**	**Yes**	**Yes**	Possibly
Fair	Fair	Low	**Yes**	Possibly	Possibly	Possibly
Fair	Fair	Med	**Yes**	**Yes**	**Yes**	Possibly
Fair	Fair	High	**Yes**	**Yes**	**Yes**	Possibly
Fair	Poor	Low	Possibly	Possibly	Possibly	Possibly
Fair	Poor	Med	**Yes**	**Yes**	**Yes**	Possibly
Fair	Poor	High	**Yes**	**Yes**	**Yes**	**Yes**
Poor	Good	Low	**Yes**	Possibly	Possibly	Possibly
Poor	Good	Med	**Yes**	**Yes**	**Yes**	Possibly
Poor	Good	High	**Yes**	**Yes**	**Yes**	**Yes**
Poor	Fair	Low	**Yes**	**Yes**	**Yes**	Possibly
Poor	Fair	Med	**Yes**	**Yes**	**Yes**	**Yes**
Poor	Fair	High	**Yes**	**Yes**	**Yes**	**Yes**
Poor	Poor	Low	**Yes**	**Yes**	**Yes**	Possibly
Poor	Poor	Med	**Yes**	**Yes**	**Yes**	**Yes**
Poor	Poor	High	**Yes**	**Yes**	**Yes**	**Yes**

Table A7 - Climatic Region Parameters (50)

Climatic Region	Mean annual days < =0°C	Average annual Precipitation, in (mm)
Wet-Freeze	> 100	>= 25 (635)
Wet-Nonfreeze	< 100	>= 25 (635)
Dry-Freeze	> 100	<= 25 (635)
Dry-Nonfreeze	< 100	<= 25 (635)

Table A8 - Traffic-Level Rating (50)

Traffic Level	ADT, vpd all lanes
Low	< 5,000
Medium	5,000 to 35,000
High	>35,000

Conditions

Joints and cracks should be sealed immediately following final cleaning and placing of bond breakers, if used. Sealing should only be done when the walls of the joint are dust free and dry, and when weather conditions meet the manufacturer's recommendations (51).

Materials Selection

<u>Sealants</u>
Many different types of sealants are available for resealing concrete pavements. The type to use depends on how much movement is expected in the pavement joints. Table A9 is reproduced from the literature (50) and includes most of the commonly used sealants, applicable specifications, the maximum extension allowed, and the approximate cost.

Table A9 - Summary of Sealant Materials (50)

Sealant Material	Applicable Specifications	Design Extension, % [a]	Cost Range, $/lb [b]
PVC Coal Tar	ASTM D 3406	10 to 20%	$1.75 to $2.75
Rubberized Asphalt	ASTM D 1190, AASHTO M 173, ASTM D 3405, AASHTO M 301	15 to 30%	$0.60 to $1.00
Low Modulus Rubberized Asphalt	Modified ASTM D 3405	30 to 50%	$0.70 to $1.20
Polysulfide (1 & 2 Part)	Fed SS-S-200E	10 to 20%	Not Available
Polyurethane	Fed SS-S-200E	10 to 20%	$5.20 to $7.20
Silicone (non-sag)	ASTM D 5893	30 to 50%	$6.50 to $9.00
Silicone (self-leveling")	ASTM D 5893	30 to 50%	$6.50 to $9.50

[a] Consult manufacturers for specific design extensions.
[b] Based on 1998 estimated costs.

Backer Rod

Backer rod is placed in the joint prior to sealing for three reasons:
1) it keeps the sealant from filling up the joint reservoir and seeping into the contraction crack beneath, which reduces cost,
2) it prevents the sealant from bonding to the bottom of the reservoir, which keeps the sealant in tension rather than combined tension and shear, and
3) it maintains a consistent sealant thickness.

Backer rod should be flexible, compressible, not shrink, not absorptive, and not reactive with the sealant. A list of several types of common backer rod are shown in Table A10.

Table A10 - Backer Rod Materials (50)

Backer Material Type	Applicable Standard	Properties	Compatibility
Extruded closed-cell polyethylene	ASTM D 5249 Type 3	NMA, ECI, NS	Most cold-applied sealants
Cross-linked extruded closed-cell polyethylene	ASTM D 5249 Type I	HR, NMA, ECI, NS	Most hot- and cold-applied sealants
Extruded polyolefin	ASTM D 5249 Type 3	NMA, NS, NG, CI, IJ	Most cold-applied sealants

CI = Chemically inert
NG = Non-gassing
ECI = Essentially chemically inert
NMA = Non-moisture absorbing
HR = Heat resistant
NS = Non-staining
II = Fills irregular joints well

Joint Reservoir Dimensions

The width and the thickness of the sealant in the joint affects performance of the seal. Therefore, there are recommended ratios of width to thickness (W:T), called the shape factor, depending on what type of sealant is used. Figure A13 shows a typical joint cross-section with backer rod and sealant in the joint and dimensions W and T. Table A11 summarizes typical shape factors for different types of sealants.

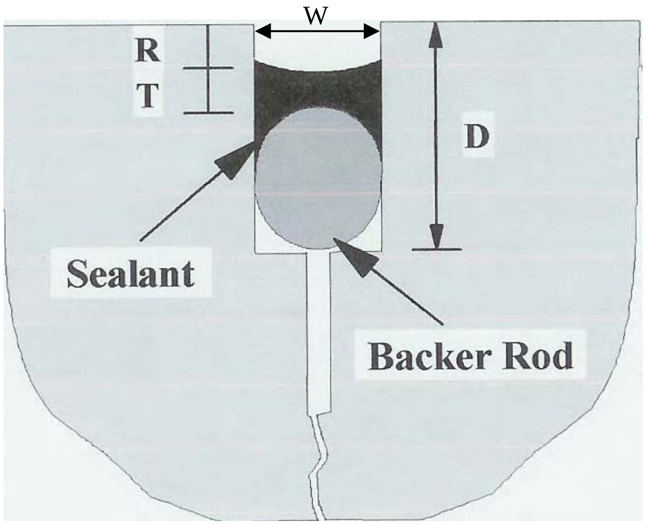


Figure A13 – Typical Joint Cross-section

Table A11 - Recommended Shape Factors.

Sealant Material Type	Typical Shape Factor (W:T)
Rubberized Asphalt	1:1
Silicone[a]	2:1
PVC Coal Tar	1:2
Polysulfide and Polyurethane	1:1

[a] minimum thickness = 6mm; maximum thickness = 13mm

In addition, the joint width should be wide enough so the sealant does not stretch more than 20 percent in winter. Therefore, the joint width is a function of joint spacing. Based on this criteria values for minimum joint width are shown in Table 10.

Table A12 - Typical joint design dimensions.

Maximum Joint Spacing, ft (m)	Minimum Joint Width, in (mm) [a]	
	Nonfreeze Region[b]	Freeze Region [c]
<=4.6	0.25 (6)	0.40 (10)
4.7 to 7.6	0.25- 0.40 (6 - 10)	0.40-0.50 (10 – 13)
7.7 to 12.2	0.40-0.50 (10 - 13)	0.50-0.75 (13 - 19)
12.3 to 18.3	0.50-0.75	0.75-1.1

	(13 - 19)	(19 - 29)

[a] Installation temperature is 81F (27°C), base is stabilized, $\%E_{max}$ <=20%.
[b] Minimum nonfreeze region temperature is 19F (-7°C).
[C] Lowest freeze region mean monthly temperature is, -15F (-26°C).

Installation

Sealant removal and replacement methods depend on several factors including: joint dimensions, hardness of existing sealant, and cleanliness of the joint after sealant removal. A flow diagram based on the description of the joint preparation and installation process in the literature (50) is shown in Figure A14 and depicts the decision process.

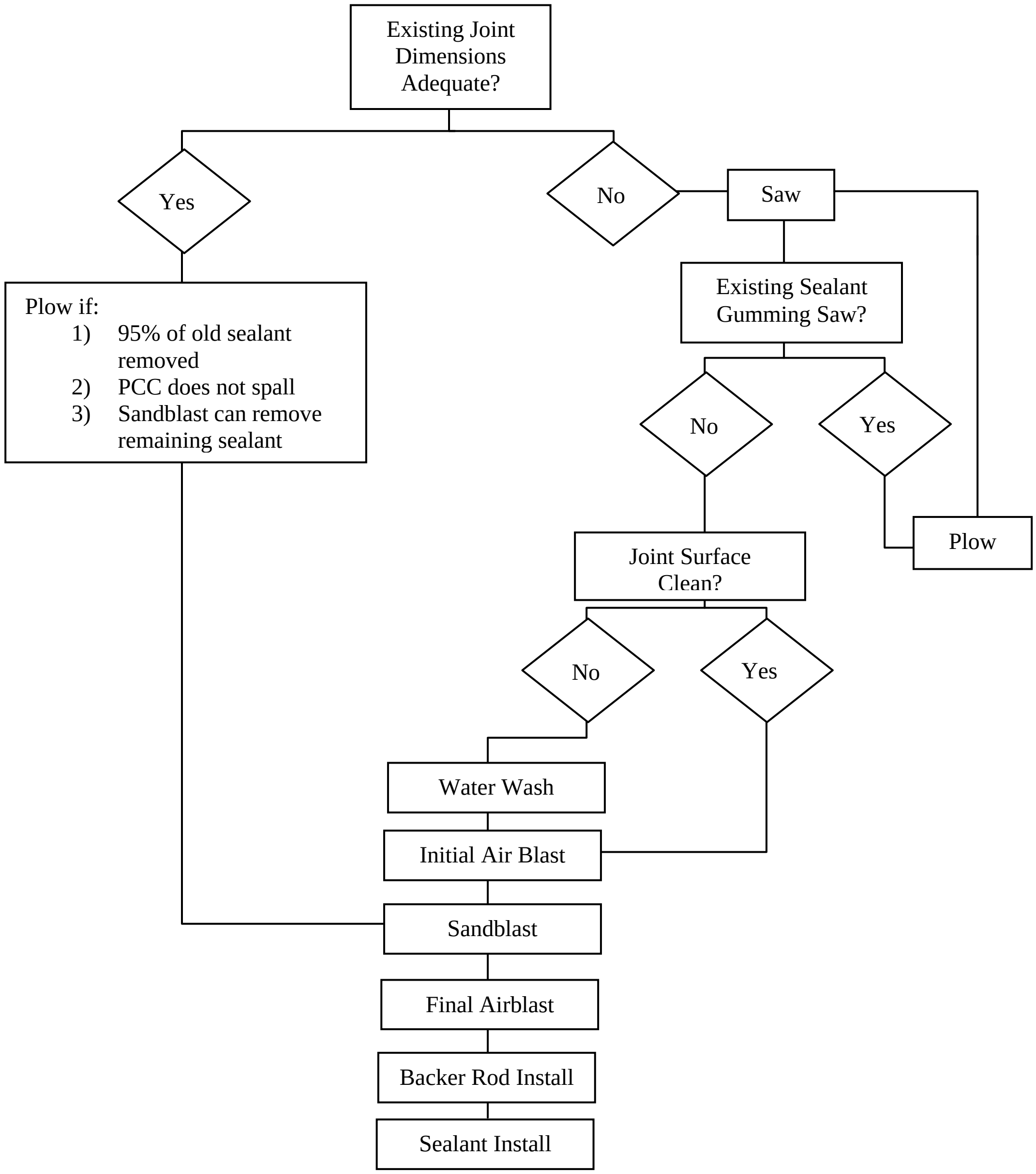


Figure A14 – Decision Process for Joint Resealing

Diamond Grinding

Diamond grinding is used to restore the surface longitudinal profile and improved ride quality of a concrete pavement. Benefits from diamond grinding include: the removal of joint crack faults and improvement of skid resistance.

Pavement Selection

The pavement should not have corner breaks, spalling or popouts. The visible surface distress may include low severity cracking, faults not exceeding 0.25 inch, and moderate to severe polishing.

Diamond grinding repairs functional deficiencies of the pavement. Structural deficiencies will require an overlay or reconstruction. Pavements with moderate to advanced material related distresses such as alkali-silica reaction or D-cracking are not good candidates for diamond grinding.

Tables A13 and A14 (52) provide a guide for determining when diamond grinding is appropriate as a function of pavement type and traffic level.

Table A13 - Trigger Values for Diamond Grinding (52)

	JPCP			JRCP			CRCP		
Traffic Volumes*	High	Med	Low	High	Med	Low	High	Med	Low
Faulting avg inches (mm)	0.08 (2)	0.08 (2)	0.08 (2)	0.16 (4)	0.16 (4)	0.16 (4)		N.A.	
IRI in/mi	63	76	90	63	76	90	63	76	90

*Volumes: High ADT>10,000; Med 3000<ADT<10,000; Low ADT <3,000

Table A14 - Limit Values for Diamond Grinding (52)

	JPCP			JRCP			CRCP		
Traffic Volumes*	High	Med	Low	High	Med	Low	High	Med	Low
Faulting avg, inches (mm)	0.35 (9)	0.5 (13)	0.6 (15)	0.35 (9)	0.5 (13)	0.6 (15)		N.A.	
IRI in/mi	160	190	222	160	190	222	160	190	222

*Volumes: High ADT>10,000; Med 3000<ADT<10,000; Low ADT <3,000

Factors which require other repairs to be made before diamond grinding include:

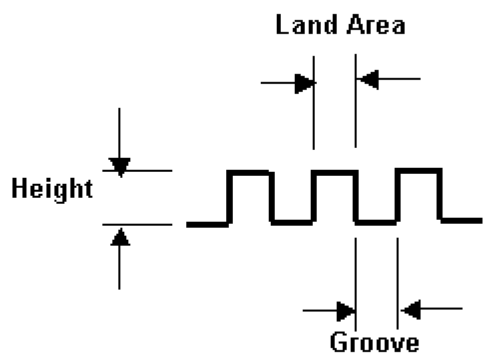
- Evidence of severe drainage or erosion indicated by severe faulting (> ¼ in) or pumping,
- The presence of progressive transverse slab cracking and corner breaks in jointed pavements.
- Joints and transverse cracks with a load transfer of less than 60 percent should be retrofitted with dowels prior to diamond grinding (see publication FHWA-SA-97-103 for more information on load transfer restoration). An effort should be made to restrict total deflection of slabs at the joints to less than 1/64-inch. Slab stabilization can be used to restrict the total deflection of slabs.
- Significant slab replacement and repair.

Conditions

Diamond grinding must not be done when the water used for lubricating the diamond grinding equipment could freeze (53).

Installation

Diamond grinding equipment should be purpose-built, self-propelled equipment for grinding concrete pavement in the longitudinal direction. The equipment should not cause undue strain or damage to the underlying surface of the pavement, cause ravels, aggregate fracture, spalls, or disturbance to the transverse or longitudinal joints. The cross-sectional pattern should conform to that shown in Figure A15.



	Range of Values mm (in)	Hard Aggregate mm (in)	Soft Aggregate mm (in)
Grooves	2.0 – 4.0 (0.08-0.16)	2.5 – 4.0 (0.1-0.16)	2.5 – 4.0 (0.1-0.16)
Land Area	1.5 – 3.5 (0.06-0.14)	2.0 (0.08)	2.5 (0.1)
Height	1.5 (0.06)	1.5 (0.06)	1.5 (0.06)
No. Grooves per meter	164 – 194 (50-60)	174 – 194 (53-60)	164 – 177 (50-54)

Figure A15 – Approximate Geometry of Diamond Grinding Cross-Section (52)

Equipment will be used to vacuum the surface of the pavement after grinding to remove excess slurry and for preventing dust from escaping into the air.

The transverse slope of the pavement shall be uniform so that no depressions or misalignment of the slope greater than 0.10 percent exists when tested with a 10 foot straightedge (53). This requirement does not apply across longitudinal joints. Adequate cross slope drainage must result after grinding so that ponding of water does not occur.

All joints shall be sealed after grinding is completed.

Cross-stitching

Pavement Selection

Cross-stitching is a preventive maintenance technique intended for concrete pavements in good condition except for the few longitudinal cracks needing repair. Cross-stitching maintains aggregate interlock at the crack or joint by providing reinforcement. Tie bars or fiberglass panels used for cross-stitching prevent the crack or joint from vertical or horizontal movement.

Installation

The cross-stitch process requires holes to be drilled in the pavement at an angle of 35 to 45 degrees from the horizontal perpendicular to the crack or joint. The holes should intersect the crack or joint at mid-slab depth. A ¾-inch deformed reinforcing bar is inserted into a 1-1/8 inch diameter hole. Holes should be drilled on 24 to 36 inch centers depending on traffic level. Heavy truck traffic requires 24 inch centers. Drills that minimize damage to the pavement should be used. Drilling debris should be removed by blowing compressed air into the hole. Epoxy resin should be injected into the holes prior to inserting the tie bars. The volume of epoxy resin injected should be the hole volume minus the bar volume. Tie bars should be inserted into the holes while the epoxy is still liquid with about 1 inch of the bar remaining above the pavement surface (54).

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