Cold Recycled Bituminous Pavement
Derby-Charleston, Vermont
Final Report

May 2007

Report 2007 - 5
Reporting on Work Plan 93-R-1

State of Vermont
Agency of Transportation
Materials and Research Section

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1. Report No. 2007-5	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle **Cold Recycled Bituminous Pavement** Derby-Charleston, VT		5. Report Date May 2007 6. Performing Organization Code
7. Author(s) Nicole Crum, Jennifer Fitch		8. Performing Organization Report No. 2007-5
9. Performing Organization Name and Address Vermont Agency of Transportation Materials and Research Section National Life Building Drawer 33 Montpelier, VT 05633-5001		10. Work Unit No. 11. Contract or Grant No.
12. Sponsoring Agency Name and Address Federal Highway Administration Division Office Federal Building Montpelier, VT 05602		13. Type of Report and Period Covered Final (1993-2004) 14. Sponsoring Agency Code

15. Supplementary Notes

Reporting on Work Plan 93-R-1, Research Report 94-6, U96-3, U96-10, U99-2

16. Abstract

This report documents the evaluation of a cold recycled bituminous pavement (CRBP). The Vermont Agency of Transportation constructed this experimental treatment along VT Route 105 in the towns of Derby and Charleston in 1993. In addition, one control section, consisting of an overlay, was applied in conjunction with the project.

Cracking, rutting, and roughness were documented on an annual basis prior to and following construction to evaluate pavement condition. These results are presented herein with recommendations on possible further research efforts on this topic.

17. Key Words Pavement Cold Recycled Bituminous Concrete	18. Distribution Statement No restrictions		
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. Pages 14	22. Price

INTRODUCTION:

With a growing number of pavements in need of reconstruction or rehabilitation and ever increasing construction costs, State Agencies are seeking out cost effective long-lasting treatments. A method known as cold recycled bituminous pavement, or CRBP, utilizes preexisting in-place bituminous pavement to construct a new bituminous layer during roadway rehabilitation. This varies from full reconstruction methods, which typically involve the removal and replacement of the existing pavement layer. The reuse of in-place materials reduces the overall cost of pavement rehabilitation by the preservation of aggregates and bitumen. Additionally, the construction of cold recycled bituminous pavement reduces the impact on the environment and conserves energy in comparison to traditional methods.

The standard CRBP process includes the reclamation of the existing pavement to a typical depth of 65 to 125 mm. The reclaimed materials are then crushed and mixed with a predetermined amount of asphalt emulsion or other binding agent. The mixed-composite material is reapplied and compacted to a specified density. The new pavement layer is allowed to cure prior to the application of a wearing surface. In most cases, the reconstruction is carried out onsite continuously through the use of a recycling train. As would be expected, the CRBP process has been shown to successfully retard reflective cracking, which is the propagation of cracks from the preexisting pavement.

In an effort to assess the performance and cost effectiveness of a cold recycled bituminous pavement in a cold weather climate, the Vermont Agency of Transportation constructed this experimental treatment along VT Route 105 in the towns of Derby and Charleston in 1993. For comparison, one control section consisting of a standard overlay treatment was applied in conjunction with the project. Pavement studies to characterize the current condition of the various treatments were conducted prior to and following construction on an annual basis. The following report summarizes the findings from annual data collection efforts and subsequent recommendations. With regards to a cost analysis, it is important to note that unlike CRBP a standard overlay only includes the application of a new wearing course but does not address the underlying pavement structure.

PROJECT DESCRIPTION:

The CRBP reconstruction occurred in 1993 along a 5.683 mile segment of VT Route 105 in the towns of Derby and Charleston. This project was completed under two separate contracts beginning at mile marker (MM) 0.800 in the town of Derby. The rehabilitation effort extended southeasterly to MM 0.800 in Charleston. All of these projects were executed as one contract for construction and research evaluation purposes. According to the construction plans, the work consisted of a 4” cold plane, cold recycling, bituminous concrete resurfacing, signs, guardrails, drainage, pavement markings, safety upgrading, and other related items.

Approximately 5.483 miles of the project received the cold in place bituminous pavement treatment in accordance with Section 415, "Cold Recycled Bituminous Pavement", of the 2001 Vermont Agency of Transportation Standard Specification for Construction. The treatment included the removal of 4" of in-place pavement which was then immediately processed and recycled using the recycle train method. Following the addition of asphalt emulsion at a specified rate of 1.24 gallons per square yard, the composite was reapplied and compacted. Please note that there is no record of the type of emulsion that was incorporated into the mix. This base was then treated with 1 ¾" of a type II binder course, which contains a nominal maximum size aggregate of 0.75", and 1 ½" of a type III wearing course, which contains a nominal aggregate size of 0.50".

In order to conduct a comparative analysis, a control section was established between MM 0.00 and MM 0.600 and 0.800 in the town of Charleston. The recycling process was not carried out within these limits. However, both overlays were applied in this location as specified in the paragraph above. Table 1, provided below, displays the limits of each treatment as well as the number of test sites identified within each section.

Derby-Charleston, STP 9248 (1), Project						
Section Type:	Number of Test Sites:	Mile Marker:	Town:	Mile Marker:	Town:	Distance (mi.):
CRBP	11	0.800	Derby	0.600	Charleston	5.483
Overlay	2	0.600	Charleston	0.800	Charleston	0.200

Table 1 – Derby-Charleston Project limits

A summary of construction and compliance test results were previously furnished within a VTrans report, 94-6, entitled, "Cold Recycled Bituminous Pavement, Derby-Charleston, VT 105." Pavement cores extracted from the CRBP section revealed that the newly constructed pavement varied from a depth of 1 ½ to 3 inches. In addition, the specification under Section 415 from 1992 does not designate a minimum compaction value for the CRBP, rather it refers to approval by the Engineer. Current specifications indicate a minimum compaction of 95% of the target density. Compaction tests performed on the recycled material yielded compaction results ranging from 92.1% to 93.4% resulting in an air void content of 7.9% to 6.6%. In comparison to the current specifications, it may be surmised that the new CRBP layer with a higher air void content would be more susceptible to freeze thaw damage as water could accumulate in the additional void spaces.

A total of 13 test sites were established throughout the length of the project. Of these 13 test sites, two sites were located within the control section (MM 0.6 to 0.8), and 11 sites were identified within the experimental section, as shown in Table 1. Each test site consisted of a length of 100' in the direction of travel and was approximately 22' wide encompassing both the east and westbound lanes. Generally, each test site was examined annually for cracking, rutting, and IRI.

HISTORICAL INFORMATION

As with any surface treatment, the overall success of a pavement is often dictated by the underlying structure. Insufficient lateral support may cause fatigue cracking or rutting. An impervious media coupled with surface cracks, allows for further water infiltration leading to freeze-thaw cracking which has been shown to compound thermal cracking. Therefore, it is important to examine the history of the surface treatment as well as the underlying soils that support the overall roadway structure. According to historical data the original construction of this highway was carried out in 1936 with an unknown "bituminous mix."

A 2.353 mile road section in Charleston between MM 0.000 and MM 2.353 was reconstructed in 1965 and received a plant mix overlay of an unknown thickness. In 1973, the road section between MM 2.233 and MM 3.723 in Derby was resurfaced with 0.625" of plant mix. In 1980, the road section between MM 3.286 in Derby and MM 0.800 in Charleston was resurfaced with a ¾" plant mix. This section was resurfaced again in 1985 with another ¾" of plant mix. The section of road between MM 0.601 and MM 2.051 in Derby was resurfaced with a ¾" plant mix in 1982, and the section between MM 0.667 and MM 3.286 in Derby was resurfaced with a 1" plant mix in 1984.

According to the Natural Resources Conservation Service (NRSC), the soils throughout the length of the project consist of a mix of Cabot silt loam, Buckland fine sandy loam, Irasburg loamy fine sand, Peacham muck, Croghan loamy fine sand, and Rumney fine sandy loam. The majority of the test sites are located in areas consisting of Cabot silt loam. This series is comprised of very deep and poorly drained soils, with moderate to slow or very slow permeability and a high potential for frost action. It is important to note that silts are fine grained, nonplastic and cohesionless, while loams are typically gritty, plastic, and easily retain water. The other soil types have similar characteristics to the Cabot series, in that they are deep to very deep and moderately to poorly drained, with moderate to slow permeability and moderate to high potential for frost action. From this information, it can be surmised that this roadway section may be vulnerable to thermal cracking.

PERFORMANCE:

Cracking, rutting, and IRI values are often utilized to assess the performance and service life of pavement treatments or in this case differing rehabilitation efforts. It has been shown that the surface condition of a pavement is directly correlated to its structural condition and is a non-linear system that can be characterized by different rates of deterioration. The following is an examination of the surface condition of both the experimental and control pavements.

CRACKING

There are several causations for cracking in flexible pavements, including inadequate structural support such as the loss of base, subbase or subgrade support, an increase in loading, inadequate design, poor construction, or poor choice of materials. For this analysis, longitudinal, transverse and reflective cracking were examined. Longitudinal cracks run parallel to the laydown direction and are usually a type of fatigue or load associated failure. Transverse cracks run perpendicular to the pavement's centerline and are usually a type of critical-temperature failure or thermal fatigue that may be induced by multiple freeze-thaw cycles. Reflection cracks occur from previous cracking that may exist within the base course, subbase or subgrade material and continue through the wearing course. In all cases, the cracks allow for moisture infiltration and can result in structural failure over time.

Pavement condition surveys of each test section were conducted throughout the study duration period, with the exception of 1999, in accordance with the "Distress Identification Manual for the Long-Term Pavement Performance Program" published in May of 1993 by the Strategic Highway Research Program (SHRP). Crack data is collected by locating the beginning of ech test section, often keyed into mile markers or other identifiable landmarks. The test section is then marked at intervals of ten feet from the beginning of the test section for a length of 100'. Pavement surveys start at the beginning of the test section and the locations and length of each crack are hand drawn onto a data collection sheet. Once in the office, the information is processed and the total length of transverse, longitudinal, centerline and miscellaneous cracking is determined for each test section and recorded into the associated field on the survey form. For this analysis, failure criterion is met when the amount of post construction cracking is equal to or greater than the amount of preconstruction cracking.

I. Fatigue Cracking

The following assessment began with examining longitudinal or fatigue cracking. As indicated by the "Distress Identification Manual", fatigue cracking occurs in areas subjected to repeated traffic loading, or wheel paths, and may be a series of interconnected cracks in early stages of development that progresses into a series of chicken wire/alligator cracks in later stages. For this investigation, the wheel paths were determined to be three feet in width with the center of the left wheel path 3.5' from the centerline and 8.5' from the shoulder for the right wheel path on either side of the roadway. An important parameter considered during the pavement design process is a wheel load characterized as an ESAL, or equivalent single axle load. An ESAL is defined by Clemson University as "the effect on pavement performance of any combination of axle loads of varying magnitude equated to the number of 80-kN (18,000-lb.) single-axle loads that are required to produce an equivalent effect." Basically, pavements are designed to structurally support traffic loads which are often calculated by AADT or ESALs with regards to roadway use. ESAL information was not available for this investigation. A comparison between average cumulative fatigue cracking of the experimental and control sections vs. AADT is provided in Figure 1 below. Averages were calculated by adding up all of the recorded linear feet of cracking of each test section within one of the two treatment types and dividing by the total number of test sections. The AADT values varied between the sections, as five experimental test sites were located in an area with a higher AADT, and six experimental test sites along with two control test sites were located in an area with a lower AADT. These AADT values are reflected in Figure 1.

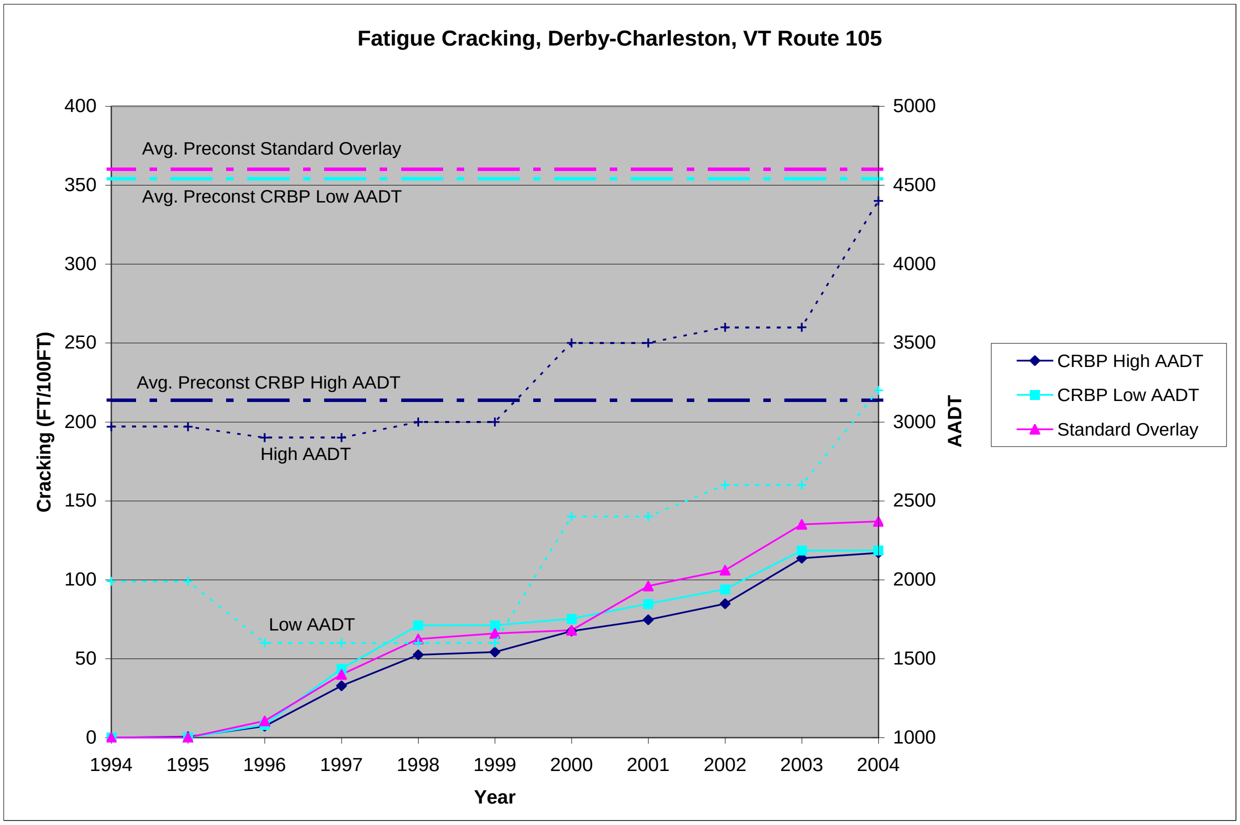

Figure 1- Fatigue Cracking

As shown within Figure 1, the graph contains a comparative analysis of fatigue cracking between the control and experimental sections with consideration to traffic volume. The traffic volume within the low AADT sections was found to be consistent for both the standard overlay and CRBP sections with a 36 percent increase in traffic within the high AADT section. As previously discussed, AADT is an important parameter when defining the traffic loading across a pavement. While the AADT only describes the average number of vehicle passes a day, it may be surmised that additional vehicles will include additional truck traffic or heavier weighted vehicles resulting in an increase in fatigue cracking. However, this assumption is not verified when assessing preconstruction fatigue cracking as the higher AADT locations appear to have less cracking as compared to the lower AADT locations. Additionally, the onset and rate of fatigue cracking does not appear to be a function of the AADT or treatment type as the amount of fatigue cracking is consistent for all test sections. Conversely, given consideration to the amount fatigue cracking in reference to preconstruction conditions, the experimental test sections within the high AADT location reached 55 percent of preconstruction cracking by 2004 while the experimental and control sections within the low AADT area were found to be 33 and 38 percent of their preconstruction values, respectively. It should be noted that preconstruction values of fatigue cracking could not be found for the test sites between MM 2.84 and MM 4.38 in Derby due to the complicated nature of the cracking, so the average preconstruction cracking values for the CRBP High and Low AADT sections could actually be higher than the displayed values.

Overall, the results from the graph do not confirm the theory that a higher AADT leads to a higher rate of fatigue cracking. The onset and rate of fatigue cracking is very similar throughout the life of the project in all test sections, and the CRBP high AADT section actually had the lowest values of fatigue cracking throughout the observation period. As a final aside, there does not appear to be a correlation between an increase in traffic volume and an increase in fatigue cracking. This may indicate that while the amount of traffic increased, the amount of truck traffic may have remained constant. Additional examination of the traffic stream is warranted.

II. Transverse (Thermal) Cracking

The formation of transverse cracking is largely due to climatic conditions and is often induced by freeze-thaw cycles or maximum low temperature shrinkage cracking. Transverse cracking of asphalt pavements is a predominant problem in New England because of the cold winter climate and many freeze-thaw cycles. In addition to comparison of the cumulative transverse cracking between the experimental and control sections, monthly average minimum temperatures were attained from a weather station in Burlington, VT, and are provided in Figure 2. Unlike AADT, temperature remains a constant variable across all test sections.

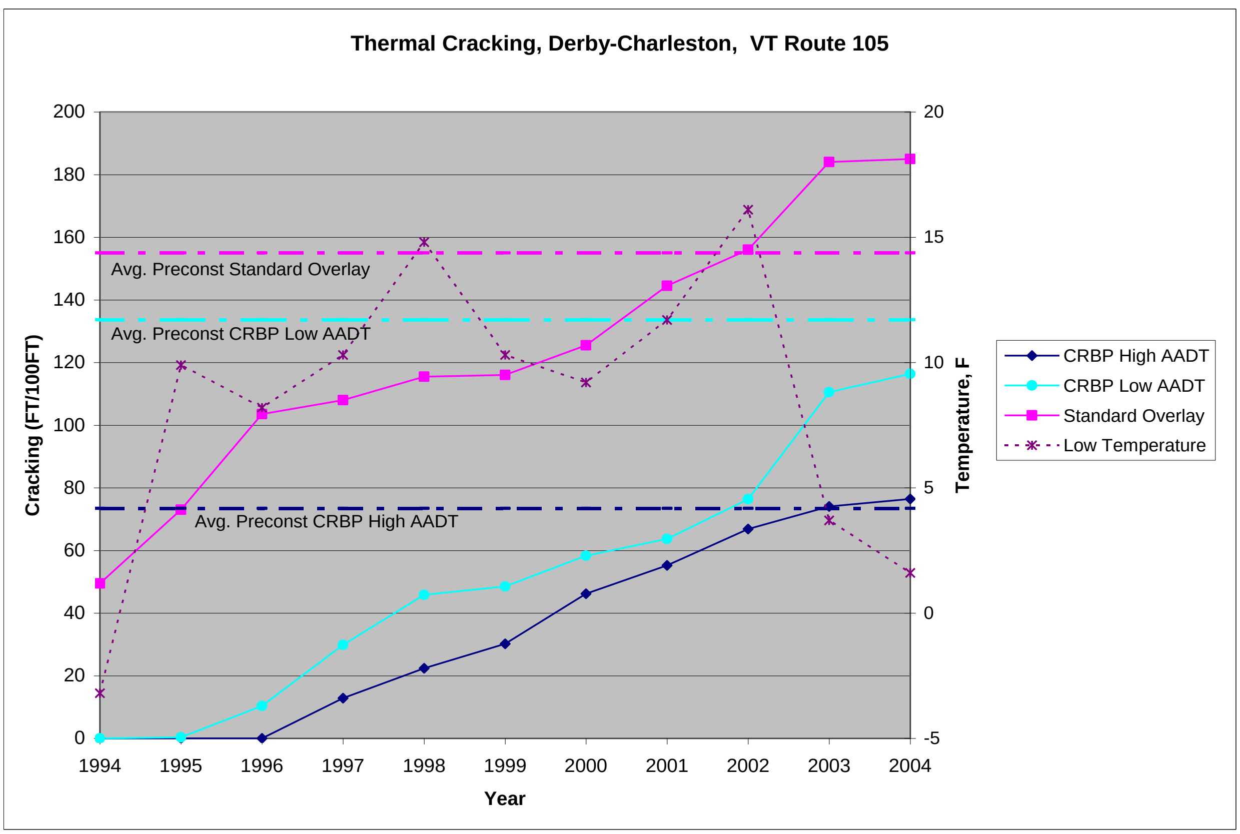

Figure 2 - Thermal Cracking

While the average amount of preconstruction cracking is variable across all sections, the minimum amount of preconstruction cracking was identified within the CRBP high AADT section at an average of 73 feet per 100 foot test section and the maximum amount of thermal cracking was found in the standard overlay section at an average of 155 feet per 100 foot test section. The onset of thermal cracking did not occur within the experimental sections until 1996, while thermal cracking was apparent within the control section in 1994, the first year following construction. While it appears that the experimental section outperformed the control section, it is interesting to notice that the rate of thermal cracking was similar between all sections. As would be expected, the standard overlay control section was the first section to meet its preconstruction level sometime between 2001 and 2002. The CRBP High AADT section met its preconstruction level sometime between 2002 and 2003, and as of 2004 the CRBP Low AADT section had not met its preconstruction level of thermal cracking.

As stated with the "Project Description" section, the CRBP layer contained an average air void content of 7.25% immediately following construction. For comparative purposes, the typical air void content of a constructed HMA (hot mix asphalt) layer varies between 5 to 6%. This indicated that the CRBP layer should have been slightly more permeable than the under and overlying HMA layers making it susceptible to saturation resulting in freeze thaw damage. Conversely, the control section should have been less susceptible to thermal cracking as it most likely contained less air voids throughout the entire matrix. However, it also contained the greatest amount of preconstruction thermal cracking. While standard overlays seal the surface of the preexisting pavement, preventing water infiltration and furtherer pavement distress, they are only expected to provide a service life of 5 to 8 years and are subject to low temperature cracking according to New York Department of Transportation. Given these properties, the overlay has performed well with consideration to thermal cracking. Overall, it does not appear that the experimental section provided a greater reduction in thermal cracking.

III. Reflective Cracking

According to Dr. Beatriz Martin-Perez of the National Research Council of Canada, reflective cracking is defined as "the propagation of cracks from the existing pavement into the layer of pavement added (overlay) during rehabilitation." As stated above, the project included the reclamation of preexisting pavement to a depth of 4". Since this process involves the removal of the preexisting pavement it is less likely to observe reflective cracking with a reclaimed stabilized base as compared to a standard overlay.

Reflective cracking was deciphered by overlaying the preconstruction data on top of the post construction data and counting the length of cracks that appear to be similar in location and overall length. However, there is a great deal of variability within the pavement surveys due to the nature of the data collection process, typically involving a large variation in field personnel, who may have differing personal interpretations. The following graph contains the average amount of reflective cracking observed on an annual basis for each test section. In addition to Figure 3 displaying the propagation of total reflective cracking over time, Table 2 provides a comparison between preconstruction conditions in association with reflective cracking.

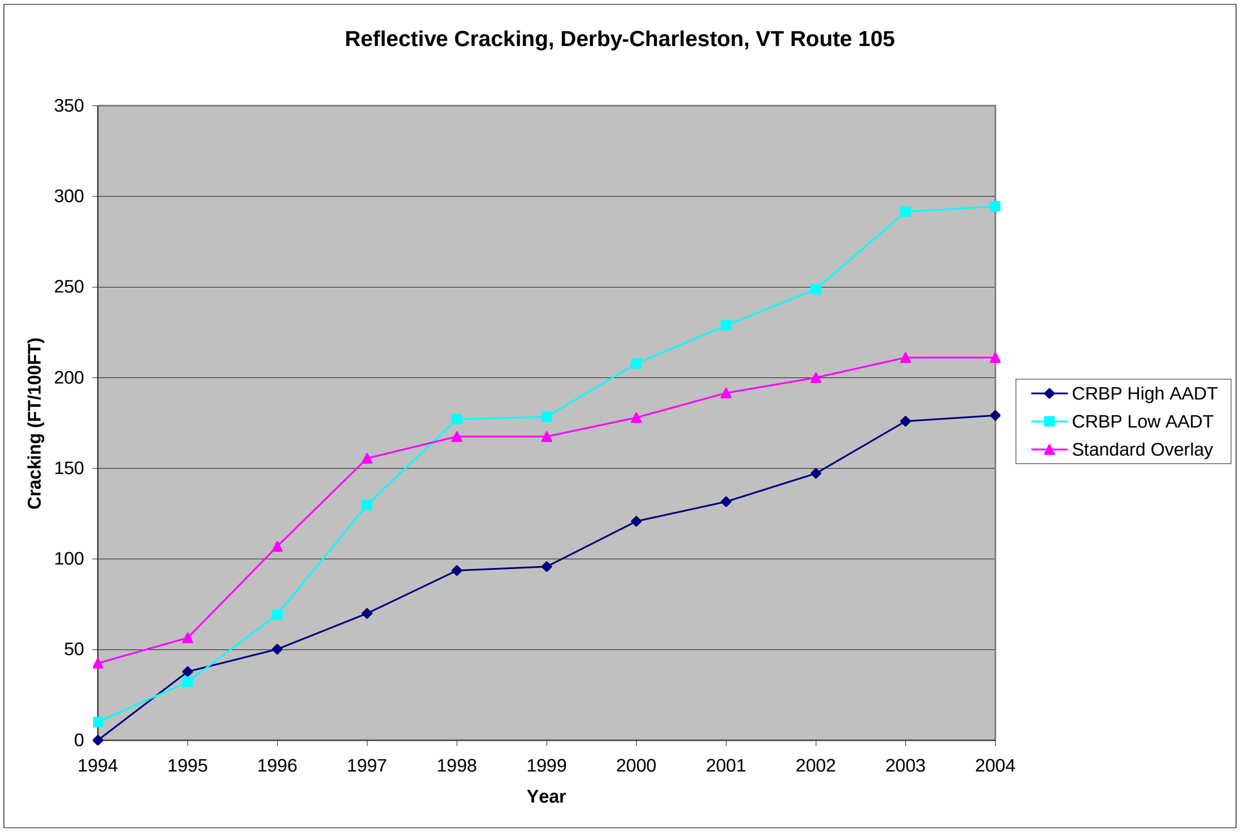

Figure 3 - Reflective Cracking

Test Site Location	Treatment Type	Total Preconstruction Cracking, LF	Total Amount of Reflective Cracking in 2004, LF	Percentage of Preconstruction Cracking
MM 1.00	CRPB	554	183	33%
MM 1.27	CRPB	434	207	48%
MM 1.40	CRPB	411	184	45%
MM 2.00	CRPB	309	127	41%
MM 2.84	CRPB	1034	195	19%
MM 3.32	CRPB	1240	324	26%
MM 3.80	CRPB	1520	374	25%
MM 4.17	CRPB	1366	-	-
MM 4.36	CRPB	1180	-	-
MM 4.60	CRPB	410	188	46%
MM 0.36	CRPB	830	292	35%
MM 0.63	Overlay	405	205	51%
MM 0.71	Overlay	666	217	33%
			Average CRBP:	**35%**
			Average Overlay:	**42%**

Table 2 - Comparison between Preconstruction and Reflective Cracking

As stated above, a greater amount of reflective cracking was expected within the overlay sections as compared to the CRBP sections. In accordance with Table 2, there does appear to be a slight increase of reflective cracking within the control sections. However, when the annual amount of reflective cracking was plotted with consideration to traffic volume, the results displayed an increase of reflective cracking within the CRBP sections under lower traffic loading. This is somewhat deceiving as the average amount of total cracking within the low AADT experimental section is almost twice that of the high AADT experimental section. Additionally, the total amount of reflective cracking within the low AADT experimental section is 294 LF on average and an average of 179 LF within the high AADT experimental section. Please note that the amount of reflective cracking in 2004 could not be deciphered for MM 4.17 and MM 4.36 due to extremely complicated cracking patterns, so these test sites were omitted from the graph and the average percent of cracking. It is interesting to note the shape of the onset and rate of reflective cracking within the experimental sections following five years of service as it appears to be consistent throughout the entire CRPB section. This pattern is furtherer supported in Figure 1 as there is an inconsistent traffic loading pattern during the first five years following construction at which time the traffic volume, although different in magnitude, maintains the same trend for both high and low traffic loading throughout the duration of the project..

RUTTING

Rutting is generally caused by permanent deformations within any of the pavements layers or subgrade and typically results from consolidation or lateral movement of the materials due to traffic loading. Throughout the duration of the investigation a rut gauge was utilized to quantify the overall depth of rut within each test section. This was done by collecting rut measurements at 50’ intervals from the beginning to the end of each test section. The measurement was collected by extending a string across the width of the road and measuring the vertical length between the string and the deepest depression within all wheel paths identified along the length of the string. All measurements were recorded onto a standard field form in 1/8” intervals. It is important to note that this procedure is highly subjective due to the nature of the data collection procedure. Table 3 displays the rut data that was collected through the duration of the investigation. All rut data is provided in Appendix A.

Average Rutting Readings for VT Route 105, Derby--Charleston								
	WB Outside WP		WB Inside WP		EB Inside WP		EB Outside WP	
Year	CRBP	STD	CRBP	STD	CRBP	STD	CRBP	STD
Precon.	0.277	0.250	0.277	0.208	0.277	0.271	0.527	0.646
1995	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1996	0.011	0.000	0.011	0.063	0.000	0.000	0.057	0.021
1997	0.034	0.063	0.064	0.063	0.034	0.021	0.159	0.021
1998	0.095	0.021	0.106	0.083	0.076	0.042	0.148	0.021
1999	0.110	0.083	0.049	0.146	0.027	0.063	0.140	0.104
2000	0.227	0.147	0.197	0.292	0.110	0.167	0.398	0.125
2001	0.269	0.167	0.246	0.292	0.174	0.167	0.417	0.167
2002	0.318	0.147	0.216	0.313	0.129	0.188	0.485	0.167
2003	0.360	0.208	0.337	0.354	0.223	0.229	0.534	0.208
2004	0.337	0.272	0.337	0.354	0.216	0.208	0.504	0.208
Precon. %	**130**	**83**	**122**	**170**	**81**	**85**	**101**	**32**

Table 3 - Rutting

In general, the overall depth of rutting increases throughout all test sections on an annual basis. However, some of the data from 2004 appears to be erroneous as the depth of rut decreases significantly in some of the test locations. According to the project history extracted from the "Pavement Management Database", there was no record of a "rut fill" at any point during the investigation period. Therefore, this data was excluded from the subset

In examining the data sets, the CRBP section displayed a greater amount of rutting than the standard overlay section at an average of 108 percent of preconstruction rutting across the full lane width as compared to 93 percent within the referenced experimental sections. Overall, this is somewhat significant at a 15 percent increase in rutting within the experimental section and may have been caused by inadequate compaction of the CRBP layer. However, given the magnitude of the percentages, no conclusions may be drawn regarding a comparison of performance. Another important observation concerns the greater amount of rutting within the inner wheels paths as compared to the outer wheel paths at 114 and 87 percent of preconstruction rutting for both the experimental and control sections, respectfully. Typically, additional consolidation would be expected to occur under the outer wheel paths resulting from reduction in structural support. While the experimental section outperformed the control section on the inner wheel path at 101 and 127 percent of preconstruction, respectfully, the opposite was true for the outer wheel path, with the experimental section reaching 116 percent of preconstruction cracking compared to only 58 percent in the control section. Additionally, the deformations within the eastbound lane reached an average of only 75 percent of preconstruction cracking, while the westbound lane had reached 126 percent of preconstruction cracking. This is an increase of more than 50 percent in the westbound lane over the eastbound lane, which may be attributed to a dissimilar traffic stream.

An investigation into the traffic pattern was completed and found that, from the available data, there were actually 624 heavy trucks in the eastbound lane compared to only 567 heavy trucks in the westbound lane in the time period between July 12 and July 26, 2000, and the eastbound lane also had more traffic at 19251 total vehicles compared to only 18306 total vehicles for the westbound lane. While this data fails to explain the phenomenon, it is from such a limited time period that it may not be representative of the typical truck traffic pattern for the section.

IRI

IRI, or International Roughness Index, is utilized to characterize the longitudinal profile within wheel paths and constitutes a standardized measurement of smoothness. According to Better Roads Magazine, “the pavement’s IRI in inches per mile measures the cumulative movement of the suspension of the quarter-car system divided by the traveled distance. This simulates ride smoothness at 50 miles per hour.” IRI values were collected on an annual basis, with the exception of 1999, through the Pavement Management Section of VTrans utilizing road profilers. Please note that the data was collected by different vendors through the investigation which resulted in poor correlation between collection events. It is important to note that the length of experimental section was 5.483 miles long as compared to 0.200 miles for the control section. IRI data is collected in increments of a tenth of a mile. Therefore, significantly more IRI data was collected from the CRBP section resulting in a valid data set. Figure 4 displays the IRI data for both the experimental and standard overlay sections.

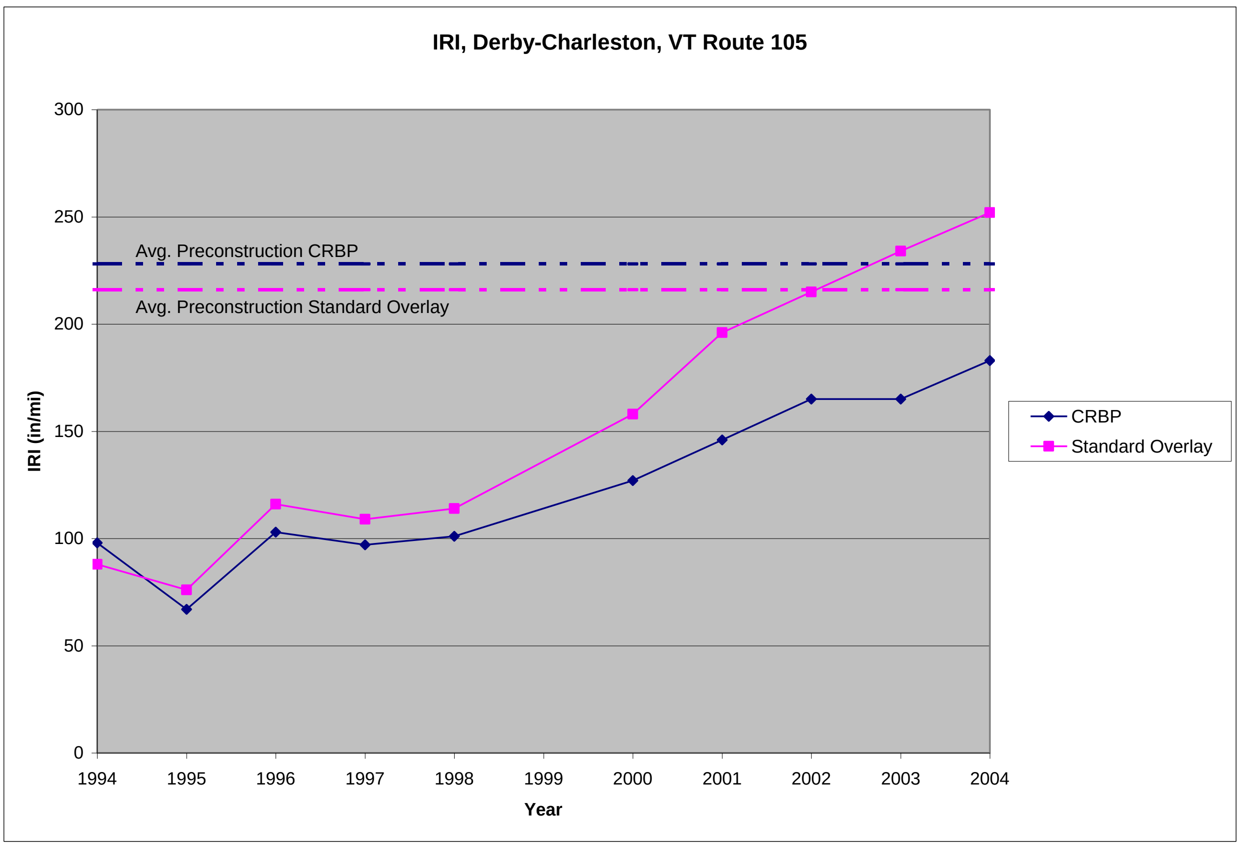

Figure 4 - IRI
(No readings taken in 1999)

There are some discontinuities within the data set. Usually IRI values are at a minimum immediately following construction as the pavement condition is optimum and will then degrade over time. Therefore, it was anticipated the there would be an upward trend throughout the years of data collection. However, in this project the IRI values fluctuated for the first few years before settling into the pattern of increasing values. The initial IRI values in 1994 are higher than those from 1995. Additionally, the IRI values collected in 1996 are greater than those from 1997, These discrepancies are most likely caused by a variation in testing equipment and calibration methods. It may also be a response from the underlying pavement condition due to frozen conditions increasing the IRI values. However, all IRI values were collected from June through August when the underlying structure would not be subjected to freezing conditions.

There is a general upward trend in the IRI values between 1997 and 2004 for both the experimental and control sections. The IRI values collected from the from the standard overlay section were found to increase at a higher rate as compared to the CRBP experimental section. In 2004, the average IRI value for the CRBP experimental section was found to be well below its preconstruction value, but the standard overlay control section exceeded its preconstruction value sometime between 2002 and 2003. In addition, the CRBP section had only reached 80 percent of its preconstruction value in 2004 as compared to 116 percent in the standard overlay section. It is important to keep in mind that IRI is directly related to all pavement distresses, which supports the data previously presented. The standard overlay control section consistently displayed a greater amount of thermal and reflective cracking than the CRBP experimental section, which would result in higher IRI values in the control section than in the experimental section.

There may be inconsistencies in the data reported due to the variability of sampling equipment. It was documented that three different road profilers were utilized for the collection of the IRI values throughout the investigation period. Each of the road profilers vary from one another which causes discontinuities between annual data sets. It is recommended that all future IRI values are collected by the same profiling device for research projects evaluating various pavement treatments in order to provide consistency and accuracy. If this is not possible, it would be desirable to establish specific highway sections to be used for statistical correlation between the equipment in consecutive years

COST:

The costs associated with the project included emulsified asphalt at $0.47/sy, bituminous concrete pavement at $5.85/sy, and cold recycling of bituminous concrete at $1.65/sy. In 2006, a similar project was let on VT 111 in Morgan and Brighton, with the costs for the items increasing to $65.00/ton for bituminous concrete, $8.00/sy for cold recycling and $45.00 per hundred weight for asphalt emulsion.

SUMMARY:

The cold recycled bituminous pavement process utilizes the in-place bituminous pavement to construct a new bituminous base layer during reconstruction. As stated previously, this method provides a reduction in overall costs and the preservation of energy as compared to typical rehabilitation effort which includes the removal and replacement of the existing pavement layer. The CRBP process has also proven effective in mitigating the propagation of reflective cracks.

Following application of the CRBP sections and a standard overlay in 1993, pavement surveys were conducted on an annual basis. This included an examination of fatigue, thermal, and reflective cracking as well as rutting and IRI. Overall, the CRBP section outperformed the standard overlay section in regards to thermal and reflective cracking throughout the duration of the project. Conversely, the experimental section displayed a higher amount of fatigue cracking and rutting which may have resulted from insufficient compaction during the construction of the CRBP layer or the incorporation of inadequate asphalt emulsion. It was interesting to note that additional consolidation was found within the inner wheel paths and that the deformations within the westbound lane were 40 percent greater than that of the eastbound lane. After additional examination of the traffic stream, the eastbound lane was found to actually have a greater amount of truck traffic than the westbound lane, which fails to provide any reason for this phenomenon. In terms of roughness, the CRBP section outperformed the standard overlay section. The standard overlay section exceeded its preconstruction value sometime between 2002 and 2003, while the CRBP section was still at only 80% of its preconstruction value in 2004.

Given the variability of results provided through this research initiative, it is difficult to make any assertions with regards to the future performance of CRBP treatments. The results do indicate that proper compaction for optimum performance is essential and certainly cost effective as compared to traditional reconstruction methods. Additional research in this area is warranted along with construction of a control section consisting of a traditional reconstruction method. Construction techniques have certainly advanced since 1992 improving the overall performance of this experimental treatment.

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