Cold Recycled Bituminous Pavement
Troy-Newport, VT
Final Report

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Materials and Research Section

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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 Troy and Newport in 1992. In addition, one control section, consisting of an overlay, was applied in conjunction with the project.

Cracking, rutting and roughness were document 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.

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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. One 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 of the existing pavement layer and placement of a new base, binder and wearing course. The use 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 preserves energy in comparison to traditional methods.

The standard CRBP process includes the reclamation of the existing pavement to a typical depth between 65 and 125 mm (2.5 to 5 inches). The reclaimed materials are then crushed and mixed with a pre-established amount of asphalt emulsion or other binding agent. Mixing is accomplished after addition of the binder material. 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. Curing times can vary greatly. 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 be successful in the retardation of reflective cracking.

In a continuing 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 an experimental treatment of CRBP along VT Route 105 in the towns of Troy and Newport in 1992. For comparison, one control section, consisting of an 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 Troy/Newport pavement project, F 034-2(10) C/1, was constructed in 1992 and began on VT Route 105 at mile marker (MM) 5.730 in the town of Troy and continued easterly to MM 2.682 in Newport for a total length of 3.277 miles. In accordance with the plans, this project included “the rehabilitation of two separate sections of roadway, cold-mixed recycling of pavement to a depth of 4”, overlays of one and three-quarters inches and one and one-quarter inches of bituminous concrete pavement, replacement of non-standard guardrail and upgrading of existing signs and sign supports.” A second contract, F 034-2(10)S C/2, was executed for the replacement of an existing 15’ diameter culvert located at MM 2.300 in the town of Newport.

Approximately 3.077 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 as shown below in Figures 1 and 2. Following the addition of asphalt emulsion at a specified rate of 1.07 gallons per square yard, the composite was reapplied and compacted. Please note that while there is no record of the type of emulsion incorporated into the mix, a modified binder was utilized. 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 ¼” of a type III wearing course, which contains a nominal aggregate size of 0.50”.

Figure 1 – Recycle Train Screen/Size Unit

Figure 2 – Recycle Train Mixing Unit/Paver

In order to conduct a comparative analysis, a control section was established between MM 0.00 and MM 0.200 in the town of Newport. 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.

Troy-Newport, F 034-2(10) C/1, Project						
Section Type:	Number of Test Sites:	Mile Marker:	Town:	Mile Marker:	Town:	Distance (mi.):
CRBP	2	5.730	Troy	6.325	Troy	0.595
Overlay	2	0.000	Newport	0.200	Newport	0.200
CRBP	2	0.200	Newport	2.682	Newport	2.482

Table 1 – Troy/Newport Project limits

A summary of construction and compliance test results were previously furnished within a VTrans report, 94-3, entitled, “Cold Recycled Bituminous Concrete Pavement, Troy-Newport, VT 105.” According to the initial report, premature longitudinal cracking up to length of 7.5’ was observed on the left and right side of the centerline. This was most likely caused by an experimental screed attached to the paver known as an “OMNISCREED.” The equipment required constant adjusting to ensure a uniform texture across the mat and to avoid tearing the bituminous concrete. The findings of this report indicated that problems with the screed, in addition to the application of a stiffer hot mix asphalt, 75 blow Marshall Mix, resulted in preliminary longitudinal cracking. The cracks incidental to this problem were not included in the evaluation.

Pavement cores extracted prior to construction revealed that the existing pavement depth varied from 6 to 11 inches. As stated previously, only 4” of the preexisting material was processed. This combination could have resulted in an increase in reflective cracking rather than a reduction. Cores were also extracted immediately following application of the CRBP. Recovery depths averaged around 2” for the 4” of recycled material. A structural loss of -0.08 per inch of recycled material was also reported. Additionally, the specification under Section 415 from 1992 does not designate a minimum compaction value for the CRBP; rather it refers to the approval by the Engineer. Current specifications indicate a minimum compaction of 95% of the target density. Due to this discrepancy, there is no information currently available with regards to the level of compaction that was achieved during construction. However, the pavement cores yielded an average air void content of 8.8%. As a final consideration, given the cold nature of the CRBP process, reuse of in place materials, addition of asphalt emulsion and higher air void content, it may be surmised that this layer would be susceptible to freeze thaw damage as water could accumulate in the void spaces.

A total of six test sites were established throughout the length of the project. Of these six, two sites were located within the control section, and four sites were identified within the experimental sections as shown in Table 1. Each test site consisted of a length of 100’ in the direction of travel and were 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 can 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 subbase of consists of 18” of gravel. Unfortunately, there is little additional information with regards to the original construction. Most of this area received a standard overlay in the 1930’s, although little is known of the treatment type.

Historical records indicate that this area was rehabilitated twice before 1992. No information on preexisting treatment was available for the area between MM 5.73 and 5.92. The remainder of this project received a 3” surface course of bituminous concrete in 1961 and in 1979 a 1” plant mix was applied along the same project limits (See Figure 3). Unfortunately, little is known about the base course. This segment consists of a 34 foot roadway width with a standard 6’-11’-11’-6’ typical road section.

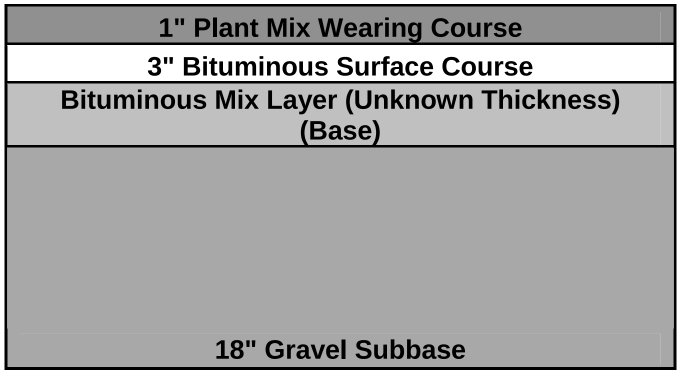

Figure 3- MM 5.92 in Troy to MM 2.632 in Newport

According to Nature Resources Conservation Service (NRSC), the soils throughout the length of the project consist primarily of Lamoine Silt. This series is defined by very deep and poorly drained soils. Permeability is moderate to moderately slow. Silts are a fine grained, nonplastic and cohesionless. Silts are also highly susceptible frost action. From this information, it can be surmised that this roadway section would be vulnerable to thermal cracking and frost action.

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. Surface condition is non-linear, 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 in accordance with the “Distress Identification Manual for the Long-Term Pavement Performance Program” published in May of 1993 by the SHRP. Crack data is collected by locating the beginning of each test section, often keyed into mile markers or other identifiable land marks. 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 a 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 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. Please note that all recorded crack data is provided in Attachment A.

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. Therefore a comparison between average cumulative fatigue cracking of the experimental and control sections vs. AADT is provided in Figure 4 below. Averages were calculated by adding up all of the recorded linear feet of cracking of each test section within one of the two mix types and dividing by the total number of test sections.

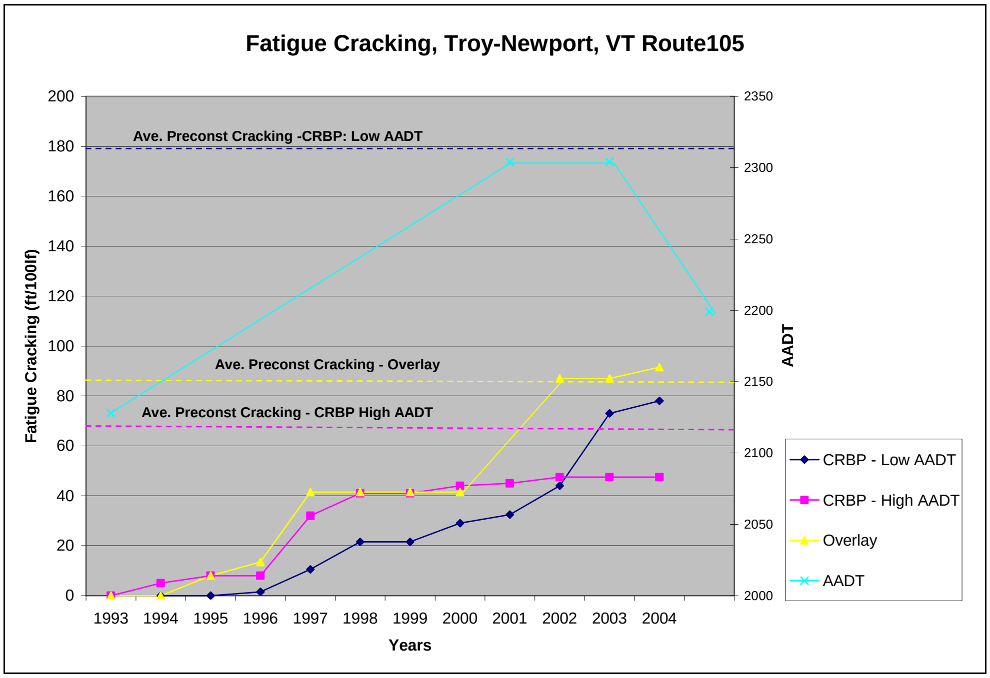

Figure-4 Fatigue Cracking VT 105 Troy-Newport

A weighted average AADT was utilized in Figure 4, otherwise defined as an average of quantities that have been adjusted by the addition of a statistical value to allow for their relative importance in a data set. However, it is important to keep in mind that the AADT, provided in Appendix B, was not a constant variable across all test sites. The AADT was relatively consistent between from MM 5.730 in Troy to MM 0.200 in Newport which contains four test sites, two in the CRBP and control sections respectively. The AADT from MM 0.200 to MM 2.682 increased by 30% as compared to previous segment. 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. This theory is proven within Figure 4 for the CRBP sections as the percentage of current fatigue cracking to preconstruction fatigue cracking is 43% and 74% within the low and high AADT sections, respectively. However, fatigue cracking within the overlay section exceeded preconstruction levels in 2000 which may be a function of the pavement treatment rather than traffic volume and composition.

It is also interesting to examine the shape of the fatigue cracking rate vs. time plot. In the case of the CRBP and standard overlay sections located within the lower AADT area, the shape of the fatigue cracking rate appear to be consistent with each other. This indicates similar pavement responses over time. Fatigue cracking from the CRBP sections with a higher AADT, exhibits a slope that is not consistent and remains relatively level for the seven years between 1998 and 2004. Additionally, the graph depicts a positive correlation between an increase in traffic loading and an increase in fatigue cracking.

II. 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. In addition to the comparison of the cumulative transverse cracking between the experimental and control sections, monthly average minimum temperatures were attained from a weather station that resides in Burlington VT, and are provided in Figure 5.

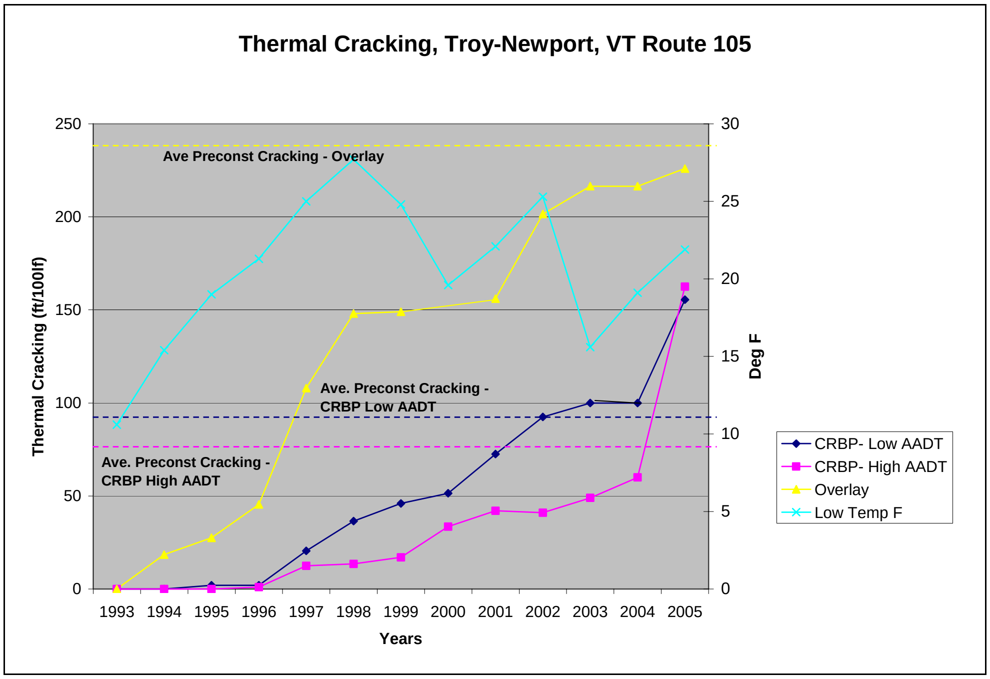

Figure-5 Thermal Cracking VT 105 Troy-Newport

Unlike AADT, temperature remains a constant variable across all test sections. While the average amount of preconstruction cracking is variable across all sections, the minimum amount of thermal cracking was identified within the CRBP low AADT section at an average of 78 feet per 100 foot test section while the maximum amount of thermal cracking was found in the overlay section at an average of 241 feet per 100 foot test section. As shown within the graph above, the overlay section outperformed the CRBP treatments within the low and high AADT sections. The CRBP section with a low and high AADT sections exceeded preconstruction levels following 9 and 11 years of service, respectively. As stated within the "Project Description" section, the CRBP layer contained an average air void content of 8.8% immediately following construction. For comparative purposes, the typical air void content of a constructed HMA (hot mix asphalt) layer varies between 4 to 6%. This indicates that the CRBP layer should have been more permeable than the under and overlying HMA layers making is 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 and accounts for the reduction in freeze thaw damage as shown in Figure 5. The shape of the rate of thermal cracking varies between all treatment types as the slopes are inconsistent from one year to the next and in comparison to each other within the same timeframe. This indicates an inconsistent pavement response across all test sites

II. 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 within the “Project Description” section above, the project typical 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 amount of reflective cracking observed on an annual basis for each test site. Please note that all CRBP sections are represented by a broken line while all control sections are denoted in a solid line. In addition to the Figure 6 displaying the propagation of total reflective cracking over time, Table 2 provides a comparison between preconstruction conditions in association with reflective cracking.

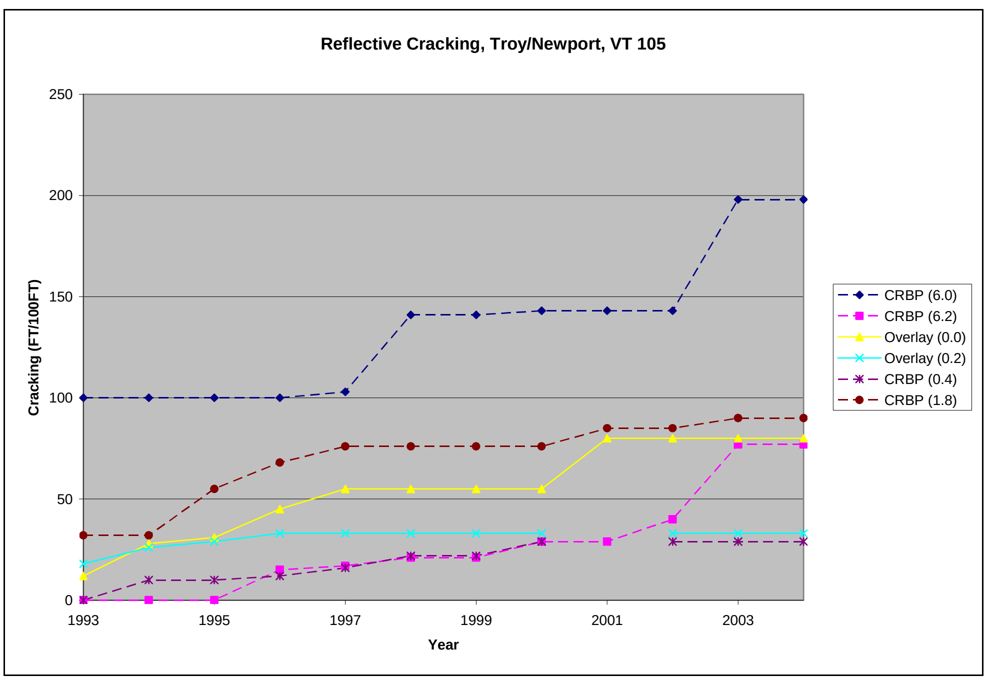

Figure-6 Reflective Cracking VT 105 Troy-Newport

Test Site Location	Treatment Type	Total Preconstruction Cracking, LF	Total Amount of Reflective Cracking in 2004, LF	Percentage of Preconstruction Cracking
MM 6.00	CRBP	468	198	42%
MM 6.34	CRBP	1388	77	6%
MM 0.00	Overlay	1414	80	6%
MM 0.15	Overlay	661	33	5%
MM 0.40	CRBP	870	29	3%
MM 1.80	CRBP	976	90	9%
			Average CRBP:	**6%**
			Average Overlay:	**5%**

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. However, on average the amount of reflective cracking within the experimental sections was found to be greater as compared to the control section. It is also interesting to note that the amount of reflective cracking within the experimental sections appears to decrease consecutively along the length of the project. This indicates that there may have been some problems related to the constructability of the CRBP sections that were resolved overtime. As stated previously, the contractor did have trouble with the OMNISCREED which resulted in the tearing of the HMA layer. In addition, the CRBP process only addressed the top 4” of the preexisting pavement layer and left the underlying pavement untouched. Any bottom up cracking that existed prior to construction would not have been addressed and most likely attributed to an increase in reflective cracking as the CRBP layer would not have as tight of a knit between the pavement layers as compared to the control section. From this information it cannot be inferred that the CRBP process reduces reflective cracking. Further consideration may need to be given to increasing the CRBP depth or increasing compaction generating a tighter knit. The results in Table 2 also depict a similar trend as shown with Figure 6. Please note that observations for TS 6.00 were not considered in the overall average for the CRBP sections as it was found to be more than three standard deviations away from the mean and is considered highly variable.

RUTTING

Rutting is generally caused by permanent deformation within any of the pavements layers or subgrade and is usually caused by 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’ foot 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. The following table displays the rut data that was collected throughout the duration of the investigation. All rut data is provided in Appendix B.

Average Rutting Readings for VT 105, Troy Newport								
	WB Outer WP		WB Inner WP		EB Inner WP		EB Outer WP	
Year	Overlay	CRBP	Overlay	CRBP	Overlay	CRBP	Overlay	CRBP
1992	0.36	0.20	0.19	0.23	0.27	0.26	0.59	0.49
1993	0.00	0.02	0.00	0.03	0.00	0.01	0.00	0.02
1994	0.00	0.03	0.00	0.02	0.00	0.01	0.00	0.01
1995	0.09	0.05	0.13	0.04	0.04	0.09	0.08	0.07
1996	0.06	0.01	0.11	0.15	0.04	0.04	0.06	0.03
1997	0.11	0.07	0.17	0.12	0.06	0.10	0.11	0.16
1998	0.13	0.06	0.15	0.06	0.09	0.10	0.13	0.16
1999	0.02	0.07	0.07	0.03	0.02	0.10	0.15	0.12
2000	0.19	0.18	0.21	0.11	0.15	0.17	0.13	0.22
2001	0.21	0.22	0.27	0.22	0.15	0.24	0.21	0.24
2002	0.32	0.24	0.30	0.19	0.17	0.20	0.17	0.22
2003	0.30	0.22	0.29	0.23	0.19	0.27	0.23	0.30
2004	0.17	0.16	0.23	0.17	0.15	0.20	0.15	0.21
Percent of Preconstruction (1993-2003)	83.10	111.39	152.63	98.92	70.37	103.81	39.32	61.73

Table 3 – Average Rut Depth Readings

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 standard overlay section displayed a greater amount of rutting than the CRBP sections at an average of 94 percent of preconstruction rutting across the full lane width as compared to 86 percent within the referenced experimental sections. Overall, this is somewhat significant at a 10 percent increase in rutting within the control section. 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 106 and 74 percent of preconstruction rutting, respectfully. Typically, additional consolidation would be expected to occur under the outer wheel paths resulting from reduction in structural support. As a final aside, the deformations within the westbound lanes are almost twice that of the eastbound lanes which may be attributed to a dissimilar traffic stream and warrants an additional examination of the traffic stream along the length of the project to identify potential causations for this phenomenon.

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. The following figure provides a summary of the IRI data:

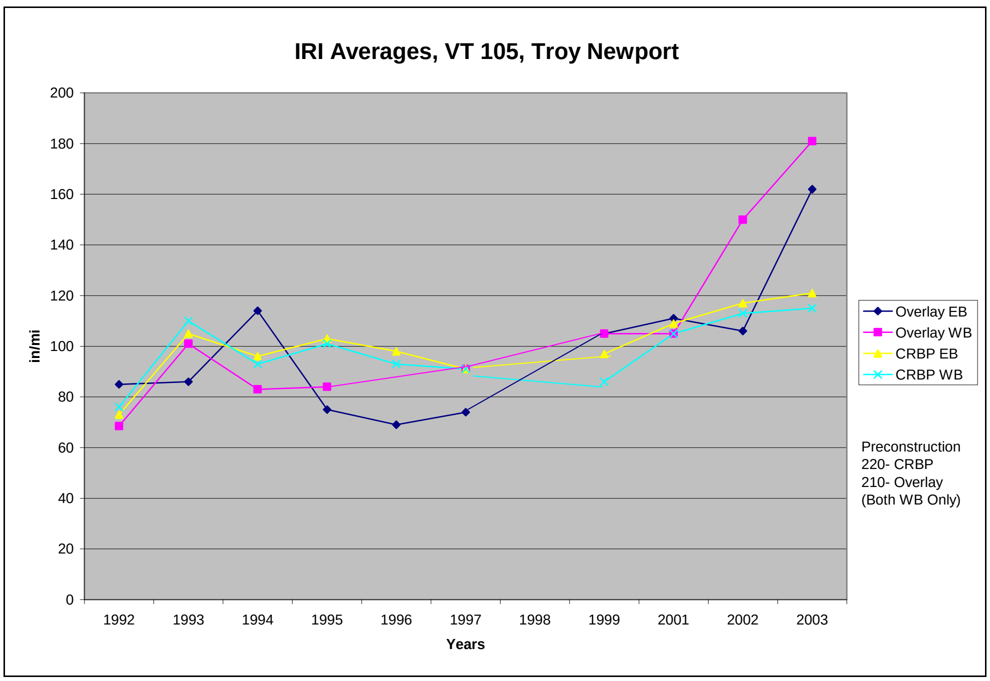

Figure 7 – IRI Averages,

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 data collection. However the initial IRI values are greater than those from following years which is most likely caused by the variation in testing equipment. 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 between 1998 and 2004. In 2004, the average IRI value for each pavement section was well below preconstruction values. This inference may not be accurate 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 investing period. Each of the road profilers vary from one another which causes discontinuities between annual data sets. The ratio between the IRI data collected in

2004 as compared to preconstruction values are fairly consistent across all test sites and were found to range between 52 percent in the westbound CRBP section to 86 percent in the overlay section. This again suggests sufficient lateral support throughout the length of the project. It is also important to keep in mind that IRI is directly related to all pavement distresses. 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

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COSTS

The costs associated with the project included emulsified asphalt at $10.50/hundred weight (CWT), bituminous concrete pavement at $32.75/ton, and cold recycling of bituminous concrete at $2.50/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 weights 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 1992, 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 CRPB sections outperformed the overlay section with regards to fatigue cracking indicating a good design with consideration to the projected traffic steam and vehicle loading. In addition, a positive correlation between larger traffic volume and increase in fatigue cracking was identified. Conversely, the experimental sections were found to have greater thermal and reflective cracking as compared to the control section. These results were most likely due to insufficient compaction and inadequate CRBP depth, respectively. The standard overlay sections displayed a greater amount of rutting than the experimental CRBP sections as would be expected given the greater amount of fatigue cracking of the referenced section. However, it was interesting to note that additional consolidation was found within the inner wheel paths and that the deformations within the westbound lane were twice that of the eastbound lane. This warrants an additional examination of traffic stream. The IRI values were fairly consistent for both treatments.

Given the variability of results provided through this research initiative, it is difficult to make any type of assertions with regards to future performance of CRBP treatments. The results do indicate that proper compaction for optimum performance is essential. It is certainly cost effective as compared to traditional reconstruction methods. Additional research in this area is warranted along with the 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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Appendix A

Crack Data for VT 105 1992-2004

CBRP1 Section								
Year	Test Site 6.0 - Troy (ft/100ft)				Test Site 6.34 - Troy (ft/100ft)			
	All Transverse Cracking	All Longitudinal Cracking	Misc. Cracking	Centerline Cracking	All Transverse Cracking	All Longitudinal Cracking	Misc. Cracking	Centerline Cracking
1992	95	269	104	90	84	284	1020	55
1993								
1993	0	21	0	100	0	0	0	100
1994	0	21	0	100	0	0	0	100
1995	0	116	0	100	4	11	13	100
1996	0	124	45	100	4	41	77	100
1997	12	135	44	100	29	88	61	100
1998	22	185	58	100	51	113	68	100
1999	39	185	58	100	53	121	68	100
2000	39	195	80	100	64	148	74	100
2001	46	202	121	100	99	161	80	100
2002	75	212	121	100	110	190	193	100
2003	75	231	121	100	125	220	203	100
2004	75	331	121	100	125	232	203	100
2005	148	426	151	0	163	380	159	100

Overlay Section								
Year	Test Site .01 – Newport (ft/100ft)				Test Site 0.15 – Newport (ft/100ft)			
	All Transverse Cracking	All Longitudinal Cracking	Misc. Cracking	Centerline Cracking	All Transverse Cracking	All Longitudinal Cracking	Misc. Cracking	Centerline Cracking
1992	391	1023	0	0	90	167	404	0
1993	0	0	0	15	2	0	0	69
1994	41	12	0	15	11	25	0	69
1995	59	17	0	15	36	38	7	69
1996	59	45	22	15	36	46	43	69
1997	74	117	49	25	51	99	99	69
1998	110	147	59	25	51	149	99	69
1999	110	149	59	25	51	149	99	69
2000								
2001	110	156	59	25	58	156	99	69
2002	116	228	21	25	86	175	132	69
2003	116	228	21	25	86	205	132	75
2004	119	228	21	25	86	205	137	75
2005	153	280	38	28	270	172	64	66

CRBP 2 Section								
Year	Test Site 0.4 – Newport (ft/100ft)				Test Site 1.8 – Newport (ft/100ft)			
	All Transverse Cracking	All Longitudinal Cracking	Misc. Cracking	Centerline Cracking	All Transverse Cracking	All Longitudinal Cracking	Misc. Cracking	Centerline Cracking
1992	114	116	640	0	41	248	687	100
1993	0	0	0	0	0	0	0	5
1994	0	27	0	0	0	6	2	5
1995	0	75	0	0	0	90	4	5
1996	2	77	18	0	0	30	4	5
1997	23	133	45	0	2	79	89	5
1998	23	167	52	0	4	99	109	5
1999	23	167	52	0	11	99	109	5
2000	34	185	61	2	33	122	109	5
2001					42	130	126	5
2002	40	200	90	3	42	155	131	5
2003	47	223	94	3	51	195	131	5
2004	62	223	94	3	58	195	131	5
2005	188	293	105	75	137	205	52	56

Definitions:

- All Transverse Cracking- this includes all cracks in the eastbound and westbound lanes, both partial and full width, perpendicular to the direction of the travel lanes.
- All Longitudinal Cracking- this includes all cracks parallel to the direction of travel for the eastbound and westbound lanes. This excludes cracking along the centerline construction joint.
- Miscellaneous Cracking- this includes all cracks other then those transverse, longitudinal, and along the centerline construction joint. This may include diagonal cracking and alligator cracking to name a few.
- Centerline Cracking- all cracking along the centerline construction joint.

Appendix B

Traffic Data

A summary of the average annual daily traffic (AADT) for the project area on VT Route 105 is presented in the table below

Town	Mile Marker	AADT			
	From To	1992	2000	2002	2004
Troy	MM 4.21 - MM 6.32	1690	1900	1900	1900
Newport	MM 0.00 – MM .20	1690	1900	1900	1800
Newport	MM 0.20 – MM 2.56	2520	2700	2700	2500

Appendix C

Rut Readings

CRBP1								
	Test Site 6.0 – Troy (in)				Test Site 6.34 – Troy (in)			
Year	Outer WB Wheel Path	Inner WB Wheel Path	Inner EB Wheel Path	Outer EB Wheel Path	Outer WB Wheel Path	Inner WB Wheel Path	Inner EB Wheel Path	Outer EB Wheel Path
1992 (Pre)	0.21	0.38	0.25	0.29	0.08	0.13	0.21	0.25
1993	0	0	0	0	0	0	0	0
1994	0	0	0	0	0	0	0	0
1995	0	0.04	0.04	0	0.042	0.042	0	0.042
1996	0	0	0	0	0.04	0.04	0.04	0.08
1997	0.08	0.08	0	0.04	0	0.13	0.08	0.13
1998	0	0	0	0.08	0.042	0.08	0.08	0.17
1999	0	0	0.13	0	0	0.08	0	0.21
2000	0.08	0.08	0.08	0.25	0.25	0.04	0.08	0.13
2001	0.25	0.25	0.17	0.25	0.29	0.21	0.08	0.17
2002	0.13	0.13	0.13	0.13	0.38	0.13	0.13	0.25
2003	0.13	0.25	0.25	0.21	0.25	0.17	0.17	0.29
2004	0.08	0.17	0.13	0.17	0.17	0.17	0.17	0.13

Control								
	Test Site 0.1 – Newport (in)				Test Site 0.15 – Newport (in)			
Year	Outer WB Wheel Path	Inner WB Wheel Path	Inner EB Wheel Path	Outer EB Wheel Path	Outer WB Wheel Path	Inner WB Wheel Path	Inner EB Wheel Path	Outer EB Wheel Path
1992	0.42	0.13	0.29	0.63	0.29	0.25	0.25	0.54
1993	0	0	0	0	0	0	0	0
1994	0	0	0	0	0	0	0	0
1995	0.13	0.13	0	0.08	0.042	0.125	0.083	0.083
1996	0.08	0.08	0	0.04	0.04	0.13	0.08	0.08
1997	0.13	0.17	0.04	0.17	0.08	0.17	0.08	0.04
1998	0.13	0.17	0.04	0.13	0.13	0.13	0.13	0.13
1999	0	0	0	0.17	0.04	0.13	0.04	0.13
2000	0.21	0.17	0.13	0.13	0.17	0.25	0.17	0.13
2001	0.21	0.21	0.13	0.21	0.21	0.33	0.17	0.21
2002	0.42	0.21	0.13	0.13	0.21	0.38	0.21	0.21
2003	0.38	0.25	0.17	0.21	0.21	0.33	0.21	0.25
2004	0.25	0.17	0.13	0.13	0.08	0.29	0.17	0.17

CRBP								
	Test Site 0.4 – Newport (in)				Test Site 1.8 – Newport (in)			
Year	Outer WB Wheel Path	Inner WB Wheel Path	Inner EB Wheel Path	Outer EB Wheel Path	Outer WB Wheel Path	Inner WB Wheel Path	Inner EB Wheel Path	Outer EB Wheel Path
1993 (Pre)	0.33	0.13	0.13	0.38	0.17	0.29	0.46	1.04
1993 (post)	0	0	0	0	0.08	0.13	0.04	0.08
1994	0	0	0	0	0.13	0.08	0.04	0.04
1995	0	0	0.13	0	0.17	0.08	0.17	0.25
1996	0	0.4	0.08	0				
1997	0.04	0	0.13	0.13	0.17	0.25	0.17	0.33
1998	0.04	0	0.13	0.04	0.17	0.17	0.17	0.33
1999	0.04	0	0	0.04	0.25	0.04	0.25	0.21
2000	0.13	0.13	0.13	0.13	0.25	0.17	0.38	0.38
2001	0.13	0.17	0.25	0.13	0.21	0.25	0.46	0.42
2002	0.17	0.21	0.17	0.13	0.29	0.29	0.38	0.38
2003	0.21	0.25	0.25	0.29	0.29	0.25	0.42	0.42
2004	0.13	0.13	0.17	0.13	0.25	0.21	0.33	0.42