REPORT 2015 - 04

VERMONT AGENCY OF TRANSPORTATION

Research and Development Section
Research Report

COLD IN-PLACE RECYCLED BITUMINOUS PAVEMENT
DORSET-DANBY, VT

Report 2015 – 04

January 2015

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Reporting on Work Plan 1996-R-8

STATE OF VERMONT
AGENCY OF TRANSPORTATION

RESEARCH & DEVELOPMENT SECTION

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16. Abstract

The report documents the long-term performance and cost effectiveness of four rehabilitation treatments in a mostly homogenous environment. The underlying subbase and subgrade soils, traffic volume, existing pavement structure and ambient conditions are similar in all sections. The Vermont Agency of Transportation specified the construction of a standard overlay, cold recycled pavement, and reclaimed stabilized base along US Route 7 in the towns of Dorset, Mt. Tabor and Danby in 1996 and 1997. Each of these treatments is intended to address various pavement distresses.

Testing and surveillance measures included annual pavement surveys prior to and following construction along with the collection of IRI reading by Pavement Management with the use of a road profiler. Sixteen 100’ test sections were established throughout the length of the project. Associated pavement surveys included the documentation of cracking and rutting. Cracking was further analyzed for total cracking, fatigue cracking and transverse cracking. Thirteen years following construction, only three of the six pavement composites display measurable amounts of fatigue and transverse cracking including the standard overlay, 3-inch CIR with 1½-inch overlay and 4-inch CIR with 1½-inch overlay. Two of the more comprehensive treatments, 4-inch CIR and 8-inch RSB with 3¼-inch bituminous overlay, displayed the greatest amount of rutting as a function of preconstruction conditions. Through the metrics developed in the study, the treatments with the thicker bituminous overlays outperformed the thinner overlays.

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ABSTRACT

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Testing and surveillance measures included annual pavement surveys prior to and following construction along with the collection of IRI reading by Pavement Management with the use of a road profiler. Sixteen 100-foot test sections were established throughout the length of the project. Associated pavement surveys included the documentation of cracking and rutting. Cracking was further analyzed for total cracking, fatigue cracking, and transverse cracking. Thirteen years following construction, only three of the six pavement composites display measurable amounts of fatigue and transverse cracking including the standard overlay, 3-inch CIR with 1½-inch overlay and 4-inch CIR with 1½-inch overlay. Two of the more comprehensive treatments, 4-inch CIR and 8-inch RSB with 3¼-inch bituminous overlay, displayed the greatest amount of rutting as a function of preconstruction conditions. Through the metrics developed in the study, the treatments with the thicker bituminous overlays outperformed the thinner overlays.

This section of roadway received a preventative maintenance treatment following year 13 of service; a greater timeframe would have been required to fully document and determine which of the treatments was the most beneficial for this section of road.

INTRODUCTION

With a growing number of pavements in need of reconstruction or rehabilitation, and ever increasing construction costs, State Agencies are seeking cost effective long-lasting treatments. Two of these methods are known as cold in-place recycling (CIR), and reclaimed stabilized base (RSB). These vary from full reconstruction methods, which typically involve the removal of the existing pavement structure and placement of a new base, binder and wearing course. CIR and RSB treatments allow for the use of in-place materials thus reducing the overall cost of pavement rehabilitation by the reuse of aggregates and bitumen. Additionally, the construction of cold in-place recycling and reclaimed stabilized base decreases the impact on the environment and conserves energy in comparison to traditional methods.

Specifically, cold in-place recycling, intended to address functional deficiencies such as block cracking and raveling, is defined by Dr. Nabil Suleiman of the North Dakota Department of Transportation (*1*) as:

> *"…a rehabilitation technique in which the existing pavement materials are reused in-place without the application of heat. The reclaimed asphalt pavement, or RAP material, is obtained by milling, planing, or crushing the existing pavement. Virgin aggregates or a recycling agent or both are added to the RAP material which is then laid and compacted*."

Typically, the pavement is reclaimed to a depth between 2½ to 5 inches. After the addition of a pre-established amount of emulsified asphalt or other binding agent, the composite material is compacted to a specified density. Curing times can vary greatly. In most cases, the reconstruction is carried out onsite continuously using a recycling train.

RSB is another cost effective long-lasting rehabilitation method typically used to address structural inadequacies such as fatigue or alligator cracking in the wheel path and rutting. Section 310 of the Vermont 2006 Standard Specifications for Construction describes RSB as (*2*):

> *"...pulverizing the existing pavement together with underlying base course material to the depth and width specified on the plans or in the contract, adding aggregate materials as required or as ordered by the Engineer, adding the stabilizing agent indicated on the plans, mixing the components thoroughly and shaping and compacting the stabilized material to the desired grade and density.*"

It is important to note that RSB recycles a portion of the subbase course as well as the existing pavement while CIR recycles only the existing pavement. Through either of these processes, the base course typically gains strength due to the addition of aggregates and stabilization agents.

In a continuing effort to assess the performance and cost effectiveness of the cold in-place recycling and reclaimed stabilized base techniques in a cold weather environment, the Vermont Agency of Transportation constructed experimental CIR and RSB treatments along VT Route 7 in the towns of Dorset, Mt. Tabor and Danby in 1996 and 1997. For comparative purposes, multiple control sections, consisting of an overlay treatment, were applied in conjunction with the project. Pavement studies to characterize the condition of the various treatments were conducted prior to and following construction on an annual basis. The following report summarizes the findings from these annual data collection efforts.

PROJECT LOCATION AND SUMMARY

The Dorset-Danby pavement project consisted of two contracts, NH 9514(1) C/1 and NH 9514(1) C/2. NH 9514(1) C/1, constructed in 1996, began in the town of Dorset at a point approximately 5.420 miles north of the Manchester/Dorset town line and extended northerly along U.S. Route 7 for a distance of 4.098 miles to MM 2.800 in the town of Mt. Tabor. Not included under this contract was the portion of U.S. Route 7 from MM 6.100 in the town of Dorset north to MM 1.000 in the town of Mt. Tabor. In accordance with the plans, work to be performed under this project included, "*cold-in-place recycling of bituminous concrete pavement, associated pavement markings, and incidental items*." The depth of the CIR treatment varied from 3 to 4 inches.

NH 9514(1) C/2, partially constructed in 1996 and completed the following year, began in Dorset at a point approximately 1.745 miles north of the Dorset/Manchester town line and extended northerly along U.S. Route 7 for a distance of 11.629 miles to the Danby/Wallingford town line ending in Danby at MM 2.473. In accordance with the plans, the project included, "*cold planing, reclaimed stabilized base construction, resurfacing of the existing highway, guard rail improvements, drainage improvements, new bridge rail, safety improvements, associated pavement markings and incidental items*." The depth of the RSB treatment was 8 inches. A summary of the pavement treatments is provided in Table 1.

It should be noted that Test Site 11 was cancelled in 2007 due to alligator cracking and Test Site 13 was cancelled in 2010 due to patched areas. Data from Test Site 11 was used for analysis prior to 2007 and Test Site 13 prior to 2010. Test Site 16 data was omitted in analysis due to minimal preconstruction cracking.

Due to the complex nature of this rehabilitation project, 16 test sites were established to document the pavement condition prior to and following construction. Test sites consisted of 100-foot sections encompassing both northbound and southbound lanes as discussed further in the "*Performance*" section. Table 2 provides the locations of each test site along with their associated treatments.

Table 1 Pavement treatment summary

Section ID	Treatment ID	MM	Town	Treatments	Contract, Year
1	1	1.745-5.420	Dorset	Leveling Course and 1½" AC Type III	2, 1997
2	2	5.420-6.100	Dorset	3" CIR 1¾" Type II Binder 1½" Type III	1, 1996 2, 1996 2, 1997
3	1	6.100-0.300	Dorset - Mt. Tabor	Leveling Course and 1½" AC Type III	2, 1997
4	3	0.300-1.000	Mt. Tabor	8" RSB 1¾" Type II Binder 1½" Type III	2, 1997 2, 1997 2, 1997
5	4	1.000-2.000	Mt. Tabor	4"CIR 1¾" Type II Binder 1½" Type III	1, 1996 2, 1996 2, 1997
6	5	2.000-2.400	Mt. Tabor	4"CIR 1½" Type III	1, 1996 2, 1996
7	6	2.400-2.800	Mt. Tabor	3"CIR 1½" Type III	1, 1996 2, 1996
8	1	2.800-2.473	Mt. Tabor - Danby	Leveling Course and 1½" AC Type III	2, 1997

Table 2 Dorset-Mt. Tabor-Danby Test site location summary

Test Site	Mile Marker	Towns	Treatment ID	Section ID
1	2.800	Dorset	1	1
2	3.200	Dorset	1	1
3	5.600	Dorset	2	2
4	5.800	Dorset	2	2
5	6.200	Dorset	1	3
6	0.000	Mt. Tabor	1	3
7	0.500	Mt. Tabor	3	4
8	0.720	Mt. Tabor	3	4
9	1.320	Mt. Tabor	4	5
10	1.600	Mt. Tabor	4	5
11	2.220	Mt. Tabor	5	6†
12	2.300	Mt. Tabor	5	6
13	2.500	Mt. Tabor	6	7††
14	2.640	Mt. Tabor	6	7
15	3.210	Mt. Tabor	1	8
16	0.000	Danby	1	8

† Test Site 11, was cancelled in 2007 due to alligator cracking
†† Test Site 13, was cancelled in 2010 due to patched areas

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 medium coupled with surface cracks allows for further water infiltration facilitating 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 roadway structure.

Historical records indicate that this area was rehabilitated numerous times prior to 1996. Gravel was placed in 1920 from MM 2.800 to MM 6.200 in Dorset, a roadway segment that encompasses TS 1 through TS 5. In 1928, a layer of surface treated gravel was applied in this same section of roadway. Due to a lack of information, the depth of the gravel is unknown. Surface treated gravel was applied in 1920 between MM 0.000 on the Dorset/Mt. Tabor line to MM 1.600 in Mt. Tabor. This encompasses TS 6 through TS 10. TS 3 at MM 5.600 in Dorset and TS 7 at MM 0.500 in Mt. Tabor all received plant mixes in 1974. The rest of the project, from MM 2.220 in Mt. Tabor to MM 2.473 in Danby, remained untouched until 1962. In the years from 1958 to 1962, the entire project length received a treatment of bituminous concrete. Following this, the project received another bituminous concrete treatment in 1982 and 1987. The preexisting pavement profiles are provided Figure 1.

According to the Natural Resources Conservation Service (NRSC) (*3*), the soils throughout the length of the project vary due to the length of the project. Test sites 1 and 2 consist of Copake and Hero gravelly fine sandy loam. This series is defined by deep and well-drained soils. Test sites 3 through 5 consist primarily of Windsor loamy fine sand. This series is defined by deep and excessively drained soils. Test sites 6 through 10 consist primarily of Hinkley gravelly loamy fine sand. This series is defined by deep and excessively drained soils. Test sites 11 through 16 consist primarily of Belgrade silt loam. This series is defined by deep and moderately well drained soils. The ground water table throughout the roadway segment is low. This, in association with the soil types and characteristics, lead this area to have low natural frost susceptibility overall.

Annual average daily traffic values for 1995 through to 2008 are shown for various sections of the project in Table 3. In general, the traffic stream has reduced over the past 12 years. The 1995 through 2005 projected equivalent single axle loads (ESALs) for these three locations, as taken from the original project plans, are as follows, 1,968,000 from Dorset MM 0.000 to MM 2.150, 1,479,000 from MM 2.150 to MM 2.900 in Mt. Tabor, and 1,450,000 from MM 2.900 in Mt. Tabor to the project limit.

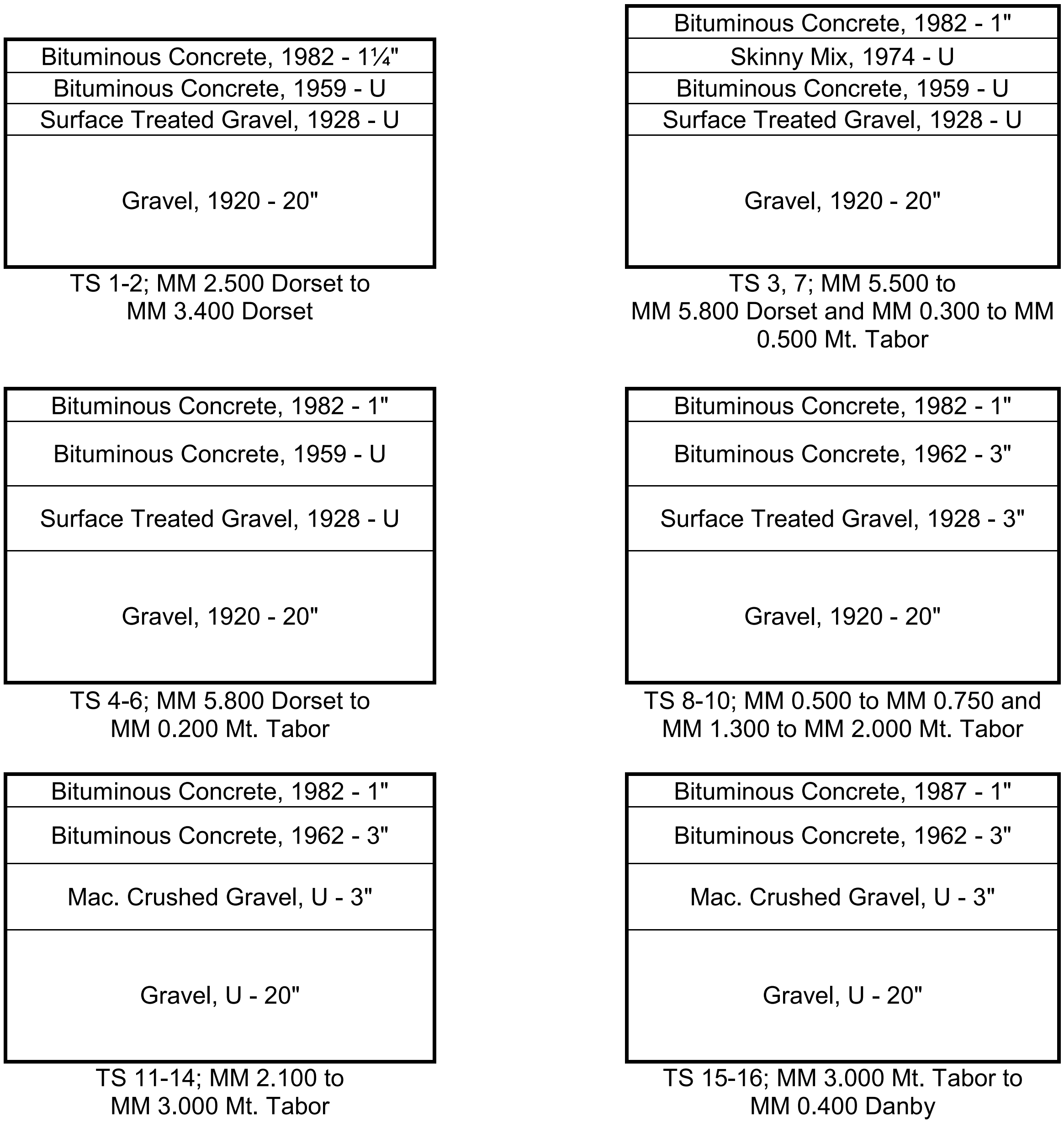

Figure 1 Test site cross-sections; U is used to denote unknown information

Table 3 Annual average daily traffic summary

Town/MM	Test Sites	1998	1999	2000	2002	2004	2006	2008
Dorset / 4.6	1-14	4900	4600	4600	4800	4600	--	4100
Mt. Tabor / 3.3	15-16	4100	--	4200	--	4300	4500	4400

The "*2008 Automatic Vehicle Classification Report*" summarizes the traffic stream and specifies the amount of traffic that is considered to be medium to heavy trucks. Medium trucks are defined as single unit trucks or FHWA vehicle class 4 through 7. Heavy trucks are tractor-trailer trucks or FHWA vehicle class 8 through 13. Two locations are listed in the Vehicle Classification Report, the first is defined as being 0.6 miles south of Emerald Lake Lane in Dorset (approximately MM 4.600), which falls between test sites 2 and 3. This location has 10.36% truck traffic compared to the AADT. The second location is 0.9 miles south of the Mt. Tabor-Danby town line (approximately MM 3.300), which falls between test sites 15 and 16. This location has 11.89% truck traffic. Therefore, the truck traffic stream appears to be comparable throughout the roadway section.

CONSTRUCTION SEQUENCING

The Lane Construction Company was awarded the project in 1996. The "*Invitations for Bids*" for these projects state, "*this contract is to be built in accordance with the 1990 Standard Specifications for Construction (4), as modified*." However, according to project records, the 1995 Standard Specifications (*5*) were actually referenced for compaction and air void requirements.

Cold-in-place recycling, shown in Figure 2, was performed during October 2006 and completed on October 10, to depths of 3 inches and 4 inches in four locations of the project. Asphalt emulsion at a rate from 1.0-1.5 gallons per square yard was added to the mix for all CIR locations. Due to seasonal constraints and sequencing of contracts, one lift of bituminous pavement was applied to all CIR locations in an effort to protect the treatment over winter months. A binder course was applied to two segments while the remaining two segments received only a wearing course. The binder course, consisting of a 1¾-inch Type II 75 blow Marshall Mix, was applied between October 23 and October 31. As stated on the approved design, the Type II mix contained 5% of performance graded (PG) 58-34 asphalt cement along with 0.5% Wetfix 312. According to historical weather data provided by Weatherunderground.com, the average ambient air temperature for this period was 47°F with an average minimum temperature of 38.5°F. A wearing course comprised of a 1½-inch Type III 75 blow Marshall Mix was placed on the remaining two CIR sections between November 6 and November 26. Similarly, this mix contained 4.8% of a PG 58-34 asphalt cement and 0.5% Wetfix 312. The average ambient air temperature for this period was 42°F with an average minimum temperature of 38°F, which is slightly elevated due to a 60°F day on November 8, 1996. The ambient air temperatures are somewhat of a concern during paving activities as they were below the specifications for placement. In accordance with the 1990 Vermont Agency of Transportation Standard Specifications for Construction, bituminous "*courses shall not be placed when the air temperature at the paving site in the shade and away from artificial heat is* *below 40°F for courses 1¼ inches or greater in compacted thickness…*" Reported ambient air temperatures are provided in Table D2 in Appendix D.

Figure 2 CIR process

Construction continued during the following summer (1997) under NH 9514(1) C/2. Miscellaneous cold planing to a depth of 2 inches was completed at various locations throughout the project for a total length of 3613 linear feet, or 0.68 miles, allowing for road variations. Bridges 52 at MM 3.501 and 54 at MM 5.019 in Dorset also received the cold planing treatments. Reclaiming activities between MM 0.300 and MM 1.000 in the town of Mt. Tabor to a depth of 8 inches were completed between July 16 and July 21. Water was used as the stabilizing agent. The reclaiming process is depicted in Figure 3 and Figure 4. Subsequently, a Type II 75-blow Marshall binder course was plaed over this roadway segment between July 19 and July 26. Finally, beginning on August 11, a 1½-inch Type III 75-blow Marshall wearing course was laid over the entire length of the project with the exception of the section between MM 2.000 and MM 2.800, encompassing TS's 11, 12, 13 and 14, in the town of Mt. Tabor where the wearing course was placed in November 1996. All paving operations were completed by September 16. It is important to note that both the binder and wearing course were consistent with those placed during the fall of 1996. Copies of mix designs can be found in Appendix E.


Figure 3 Reclaiming machine

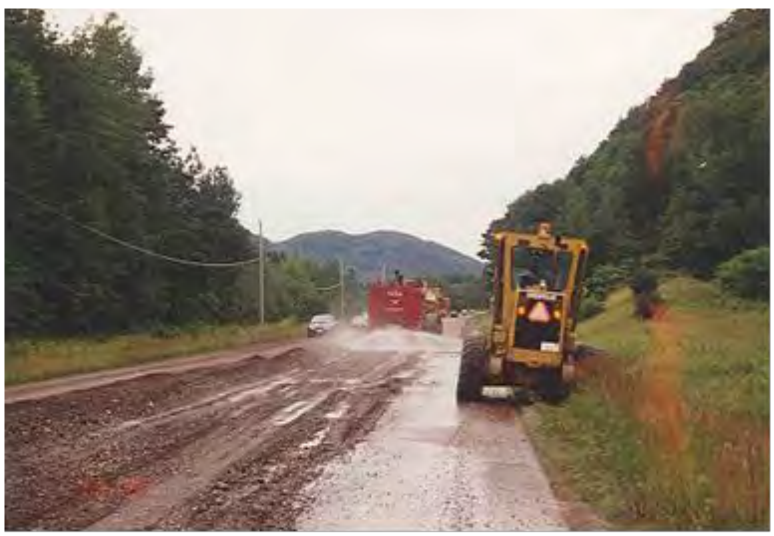
Figure 4 Water application during RSB construction

IN-PLACE TESTING DURING CONSTRUCTION

In accordance with the Agency's specifications, compaction testing was performed on each pavement layer including the reclaimed base, cold recycled pavement, binder, and wearing courses. Per the project special provisions, a minimum target density of 95% of the maximum density was to be maintained throughout the duration of the project for cold recycled pavements and reclaimed stabilized bases. As such, density testing was performed in accordance with AASHTO T 238, "*Standard Method of Test for Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth)*" (*6*). For the binder and wearing courses, the specifications state, "*the density of the compacted pavement shall be at least 92%, but not more than 96% of the corresponding daily average maximum specific gravity*." All bituminous pavement testing was performed per AASHTO T 130, Method B, "*Standard Test Method for Determining Degree of Pavement Compaction of Bituminous Aggregate Mixtures*" (*7*). All compaction results are summarized in Table 4. A copy of all compaction results is provided in Appendix F.

Table 4 Compaction summary.

Treatment	Target Density	Actual Density	Section ID 1	2	3	4	5	6	7	8
RSB	D ≥ 95%	Avg.	---	---	---	97.7	---	---	---	---
		St Dev:				2.1				
		Passing				100%				
CIR	D ≥ 95%	Avg.	---	94.8†	---	---	94.3†	No Tests	98.5	---
		St Dev:		1.9			1.5		2.1	
		Passing		40.0%			33.3%		100%	
Binder	92 ≥ D ≥ 96%	Avg.	---	94.1	---	93.2	92.3	---	---	---
		St Dev:		1.5		1.3	2.0			
		Passing		88.9%		88.9%	50.0%			
Wearing	92 ≥ D ≥ 96%	Avg.	93	93.2	93.2	93.1	91.9†	91.4†	93.4	93.5
		St Dev:	1.3	1.4	2.5	1.5	1.9	1.5	1.2	1.5
		Passing	85.7%	100%	66.7%	66.7%	50.0%	33.3%	100%	75.0%

†Values indicate that the average compaction is outside of the specifications for that treatment.

In examining Table 4, specified compaction was not achieved in many areas throughout the length of the project. Specifically, the average compaction for cold recycled pavements was found to be below 95% of the maximum density between MM 5.420 through MM 6.100 in Dorset (Treatment ID #2, TS 3 and 4) and MM 1.000 through MM 2.000 in the town of Mt. Tabor (Treatment ID #5, TS 9 and 10). Specified compaction was not achieved on wearing course between MM 1.000 through MM 2.400 in the town of Mt. Tabor (Treatment ID #5 and 6, TS 9 through 12). In almost all cases, this was due to under compaction. Therefore, these CIR segments are at a greater risk for excessive rutting over time and/or premature cracking due to construction practices. However, adequate compaction was attained on the reclaimed stabilized base layer and the majority of the binder and wearing course throughout the project.

PERFORMANCE AND OBSERVATIONS

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 varying rates of deterioration. The following is an examination of the surface condition of both experimental and control pavements.

Pavement condition surveys of each test section were conducted prior to and following construction on an annual basis 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 (*8*), with the exception of 2001 and 2005. Crack data were collected by locating the beginning of each test section, often keyed into mile markers or other identifiable landmarks. The test section was then marked at intervals of 10 feet from the beginning of the test section for a length of 100 feet. Pavement surveys initiated at the beginning of a test section and the locations and length of each crack were hand drawn onto a data collection sheet. Once in the office, the information was processed and the total length of transverse, longitudinal, centerline, miscellaneous, and fatigue cracking was determined and recorded into associated fields on the survey form. For this analysis, the failure criterion is met when the amount of post-construction cracking is equal to or greater than the amount of preconstruction cracking for each specific failure mechanism. Please note that all recorded crack data is provided in Appendix D.

Cracking

There are several causes for cracking in flexible pavements, including inadequate structural support such as the loss of base, sub-base, or sub-grade support, increased loading, inadequate design, poor construction, or poor choice of materials. For this analysis, total, fatigue and transverse cracking were examined. Total cracking is simply the total amount of cracking within a test site without regard to the type or cause of pavement distress. Fatigue cracks run parallel to the laydown direction within the wheelpaths or a small block pattern 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. In all cases, cracks allow for moisture infiltration and can result in structural failure over time.

Preconstruction Cracking Summary

Prior to construction, a site visit was performed to document the amount of cracking and rutting that was present within all test sites. Table 4 summarizes the average amount of preconstruction cracking and rutting per treatment type.

As stated previously, each treatment type is intended to address different forms of pavement distresses. RSB is intended to correct primarily structure-related deficiencies while CIR is limited to mostly functional deficiencies. Therefore, it was anticipated that the roadway segment rehabilitated with RSB would have displayed the greatest amount of fatigue cracking prior to construction when compared to the other locations. This is considering the expectation that the most comprehensive treatment would be used on the poorest section of roadway. However, this did not appear to be the case (see Table 5.) The other treatments displayed a greater amount of fatigue cracking both as a function of average linear cracking per treatment and as a percentage of total cracking. The test sites with the 3-inch CIR with 1½-inch overlay displayed similar deficiencies. It is important to note that the average amount of cracking within the 3-inch CIR with 3¼-inch overlay treatment displayed far more total and fatigue cracking and had a higher percentage of fatigue cracking to total cracking than all other segments.

Table 5 Average preconstruction pavement distress summary in LF/100 test section

Treatment	# of Test Sites	Total Cracking	Fatigue Cracking	% Fatigue to Total Cracking	Transverse Cracking	Rutting
1½" Overlay†	5	479	355	74.1%	120	0.30
3" CIR with 1½" Overlay	2	368	156	42.4%	138	0.32
3" CIR with 3¼" Overlay	2	862	680	78.9%	112	0.39
4" CIR with 1½" Overlay	2	224	155	69.2%	69	0.37
4" CIR with 3¼" Overlay	2	444	243	54.7%	115	0.26
8" RSB with 3¼" Overlay	2	349	140	40.1%	100	0.32
Overall Average:		**459**	**302**		**111**	**0.32**

†TS 16 removed

Total Cracking

Total cracking is simply the total amount of cracking within a test site with no regard to the type or cause of the pavement cracking. It is a general measure of the condition of the pavement prior to and after construction. For example, a pavement with a sufficiently greater amount of cracking prior to construction as opposed to another roadway segment can be considered to be in poorer condition comparatively. However, it is vital to determine the cause of pavement deterioration prior to selecting the most appropriate pavement rehabilitation technique. As stated previously, pavement overlays seal the roadway surface reducing infiltration, alleviating oxidation, increasing structural capacity, and correcting surface defects. The cold recycle process is intended to correct functional deficiencies such as non-wheelpath longitudinal cracking, block cracking, poor ride-ability, flushing, or raveling. Full depth reclamation is generally used to correct structural deficiencies such as fatigue or alligator cracking within the wheelpaths and rutting.

Cumulative total cracking in terms of average linear feet per treatment as a function of time is provided in Table 6 with the corresponding graphical representation in Figure 5. It is important to note that following 9 years of service, test site 11 (4-inch CIR with 1½-inch AC) was discontinued due to alligator cracking, which is indicative of deterioration and test site 13 was removed after year 12 due to the application of a shim course. All years following this were averaged amongst the remaining test sites for this treatment. For comparative purposes, the average preconstruction fatigue cracking in linear feet per treatment is also shown within the table. Through 13 years of service, only the 3-inch CIR with 1½-inch overlay has exceeded its preconstruction levels, meeting the project's definition of failure with respect to total cracking. Both treatments utilizing the thicker 3¼-inch overlay has shown only low amounts of cracking over their service life.

Table 6 Average Pavement Distress Summary in LF/100 FT Test Section

Treatment	Years											
	1	**2**	**3**	**5**	**6**	**7**	**9**	**10**	**11**	**12**	**13**	**Pre**
1½" Overlay†	0	0	13	63	72	88	253	295	343	357	394	479
3" CIR w/ 1½" Overlay	4	4	4	42	45	49	71	139	252	282	410	368
3" CIR w/ 3¼" Overlay	0	0	0	0	0	0	0	1	8	12	20	862
4" CIR w/ 1½" Overlay	0	0	0	57	57	64	127	132	258	301	324	224
4" CIR w/ 3¼" Overlay	0	0	0	0	0	0	0	22	34	42	43	444
8" RSB w/ 3¼" Overlay	0	0	0	0	13	55	106	138	213	222	224	349

†TS 16 removed

Fatigue Cracking

As indicated by the Distress Identification Manual, fatigue cracking occurs in areas subjected to repeated traffic loading, most notably the wheelpaths, and may be a series of interconnected cracks in early stages of development that progresses into a series of chicken wire/alligator cracks in the later stages. For this investigation, the wheelpaths were determined to be 3 feet in width with the center of the left wheel path 3 feet from the centerline and the right wheel path 8 feet from the centerline for both sides of the roadway. An example of fatigue cracking within test site 2 is provided in Figure 6.

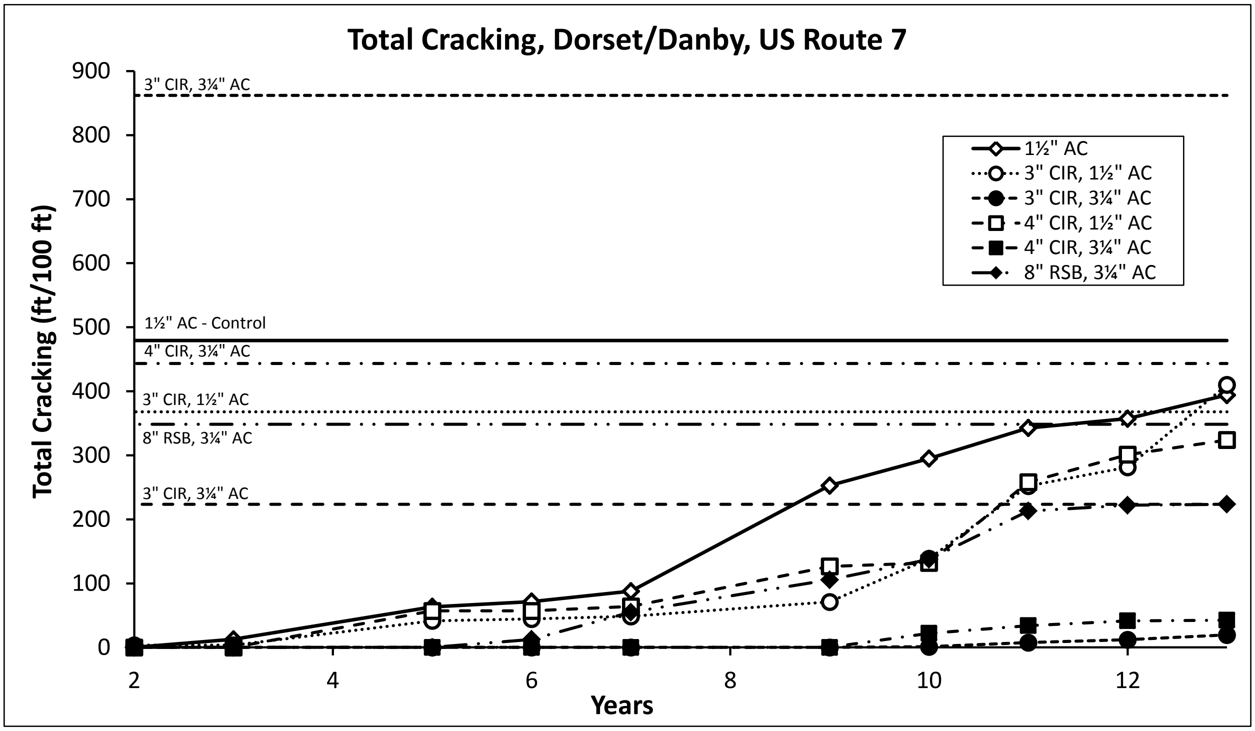

Figure 5 Total cracking in linear feet. Preconstruction values are shown as horizontal lines in corresponding line patterns.

Figure 6 Fatigue cracking (Test Site 2, year 10)

Cumulative fatigue cracking in terms of average linear feet per treatment as a function of time is provided in Figure 7. These data series consist of the averaged values of all test sites within each of the treatments. Only treatments with a substantive amount of observed fatigue cracking to date have been included. Those not included in the figure are 3-inch CIR with 3¼-inch AC, 4-inch CIR with 3¼-inch, and 8-inch RSB with 3¼-inch AC. These treatments are also excluded from subsequent cracking plots as well for identical reasons. It is important to note that after year 9, test site 11 (4-inch CIR with 1½-inch AC) was discontinued for all analyses due to alligator cracking, which is indicative of deterioration and test site 13 was removed after year 12 due to the application of a shim course. For all subsequent years, the data used were from the one remaining test site for this treatment. For comparative purposes, the average preconstruction fatigue cracking in linear feet per treatment is also shown in the figure.

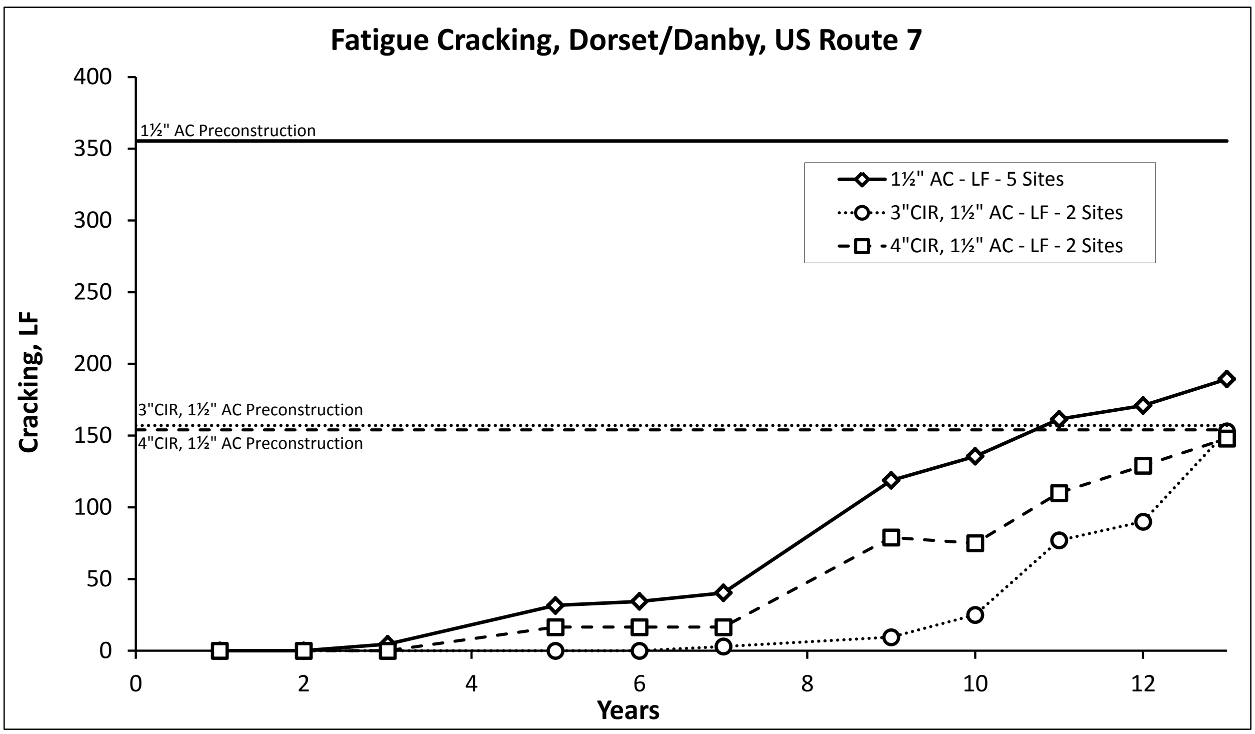

Figure 7 Fatigue cracking in linear feet, averaged for all test sites in that treatment. Preconstruction values are shown as dashed lines in corresponding colors.

As expected, sections with the 1½-inch AC overlay displayed the greatest amount of fatigue cracking as opposed to the more comprehensive rehabilitation treatments. However, it is quite impressive that current levels of fatigue cracking across all test sites have yet to meet or exceed preconstruction conditions especially with respect to the overlay only sections. Thirteen years following construction, only 53% of the preconstruction fatigue cracking on average has returned to the control treatment consisting of the 1½-inch AC. These segments are projected to meet or exceed preconstruction fatigue cracking 17 years following construction. Similarly, 97% of the preconstruction fatigue cracking on average has return to the 3-inch CIR with 1½-inch overlay treatment. These segments are anticipated to have a study life of 14 years. The 4-inch CIR with 1½-inch overlay is expected to have a 14-year study life as well. Study life expectancies were derived by producing the most appropriate projection trend lines in the year vs. cracking plots. The estimated number of years defining the life of the study is determines as the trend line surpasses the average amount of preconstruction fatigue cracking for each treatment.

Given that many of the roadway characteristics of the two CIR treatments referenced in this section are the same including the subgrade soils, preexisting pavement structure, traffic stream, and amount of preconstruction fatigue cracking, the more comprehensive treatment, or 4 inch recycled layer, was expected to outperform the less comprehensive treatment, or 3-inch recycled layer. However, as shown in the graph, this is simply not the case. The cause for fatigue cracking frequency is unknown but may be attributed to inadequate compaction of the 4-inch CIR layer. However, it is difficult to verify this assertion as no compaction tests were performed in this area.

Transverse 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 multiple freeze-thaw cycles. An example of transverse cracking is provided in Figure 8. For this analysis, any length of transverse crack was included regardless if the crack spanned from shoulder to shoulder.

Cumulative transverse cracking in terms of average linear feet per treatment as a function of time is provided in Figure 9. Only treatments with greater than 5 feet of transverse cracking per 100-foot test section 13 years following construction have been included. These include standard overlay, 3-inch CIR with 1½-inch overlay, and 4-inch CIR with 1½-inch overlay. It is important to note that following nine years of service, test site 11 (4-inch CIR with 1½-inch AC) was discontinued due to alligator cracking, which is indicative of deterioration, and test site 13 was removed after year 12 due to the application of a shim course. All years following this were averaged among the remaining test sites for this treatment. For comparative purposes, the average preconstruction transverse cracking in linear feet per treatment is also shown.

Figure 8 Example of transverse cracking (among others), within test site 1.

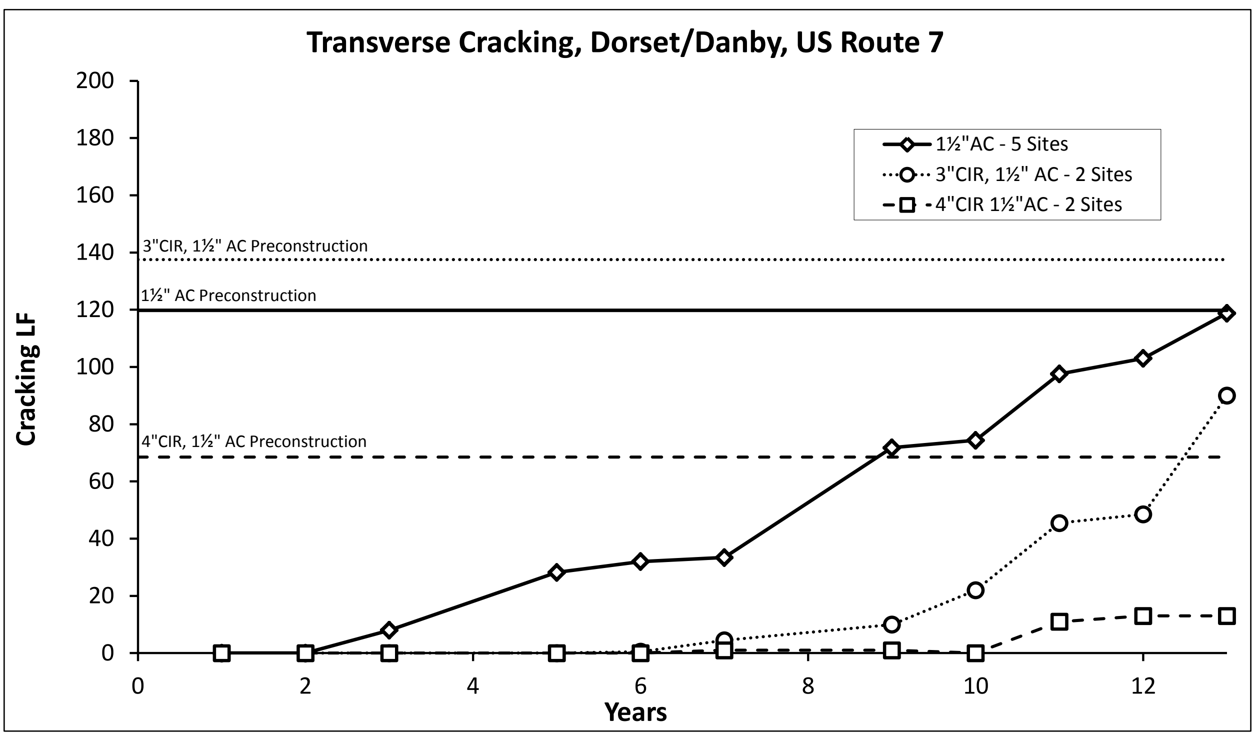

Figure 9 Thermal cracking in linear feet. Preconstruction values are shown as dashed lines in corresponding colors.

Once again, the control sections displayed the greatest amount of transverse cracking on average (118 linear feet per 100-foot test section) as compared to all other sections. Comparatively, the other two treatments were found to have 90 and 13 linear feet of cracking for the 3 and 4-inch CIR depths, respectively, 13 years following construction. In examining Figure 9, none of the treatments have met or exceeded the amount of documented transverse cracking prior to construction. As of the summer of 2010, 99% of preconstruction transverse cracking on average had returned to the control treatment (1½-inch pavement overlay). These segments are anticipated to exceed these levels 14 years following construction. In the same timeframe, 65% of preconstruction transverse cracking has returned to the 3-inch CIR treatment with 1½-inch overlay. These segments are expected to provide a 16-year study life. Similarly, 19% of preconstruction transverse cracking had returned to the 4-inch CIR treatment and these segments are anticipated to exceed preconstruction levels around 19 years of service.

Given the performance of the treatments with respect to the formation of transverse cracking and similar roadway characteristics, it is easy to surmise that the more comprehensive the treatment, the greater the service life. It is important to note however that the test sites within the 3-inch CIR with 1½-inch overlay section displayed 500% more transverse cracking as compared to test sites within the 4-inch CIR with 1½-inch overlay section. This may indicate that the pavement structure within this area is more predisposed to damage induced by freeze-thaw cycles.

Rutting

Rutting is generally caused by permanent deformation within any of the pavement layers or subgrade and is usually caused by consolidation or lateral movement (shoving) of the materials due to traffic loading. Throughout the duration of the investigation, a rut gauge was utilized to quantify the overall depth of ruts within each test section. This was done by collecting rut measurements at fifty-foot intervals from the beginning to the end of each test section. The measurements were 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 wheelpaths identified along the length of the string. All measurements were recorded onto a standard field form in ⅛-inch intervals. It is important to note that this procedure is highly subjective due to the nature of the data collection. Table 7 and Table 8 display the rut data collected for year 13 of the study. Table 7 shows the average rutting per treatment per wheelpath as of year 13 in inches, while the results shown in Table 8 are the same values represented as the percent of preconstruction rutting. All yearly rut data by individual test site is provided in Appendix C.

Table 7 Average Rut Readings at 13 Years of Service by Wheelpath (in inches).

Treatment	SB Outer WP	SB Inner WP	NB Inner WP	NB Outer WP	Averages
1½" AC	0.23	0.32	0.27	0.30	**0.28**
3" CIR 1½" AC	0.27	0.25	0.25	0.25	**0.26**
3" CIR 3¼" AC	0.08	0.38	0.23	0.25	**0.24**
4" CIR 1½" AC	0.42	0.21	0.00	0.25	**0.22**
4" CIR 3¼" AC	0.38	0.42	0.33	0.38	**0.38**
8" RSB 3¼" AC	0.48	0.31	0.38	0.33	**0.38**
Averages	**0.31**	**0.32**	**0.26**	**0.30**	**0.29**

Table 8 Average Rut Readings at 13 Years of Service by Wheelpath (as a % of Preconstruction).

Treatment	SB Outer WP	SB Inner WP	NB Inner WP	NB Outer WP	Averages
1½" AC	74%	125%	95%	88%	**95%**
3" CIR 1½" AC	86%	80%	71%	80%	**79%**
3" CIR 3¼" AC	17%	152%	74%	46%	**72%**
4" CIR 1½" AC	126%	53%	0%	60%	**60%**
4" CIR 3¼" AC	228%	155%	132%	166%	**170%**
8" RSB 3¼" AC	154%	83%	152%	99%	**122%**
Averages	**114%**	**108%**	**87%**	**90%**	**100%**

The two tables show that the trends between actual rut depth and percentages with respect to preconstruction values are mostly constant. To date, the 4-inch CIR and 8-inch RSB sections with 3¼-inch overlay display the greatest amount of rutting as a function of preconstruction conditions. As these are the most comprehensive treatments within the study, rutting may be attributed to additional consolidation of the pavement layers following construction. With respect to density testing performed during construction, all lifts within the RSB treatment were found to meet our specifications while specified compaction was not achieved on CIR and wearing course layers. All other rut percentages appear to be reasonable through 13 years of service. Overall, averaging all treatments and wheelpaths together results in 100% of rutting regained after the 13-year period.

IRI

IRI, or International Roughness Index, is utilized to characterize the longitudinal profile within wheelpaths and constitutes a standardized measurement of smoothness. According to AASHTO R 43M (*9*), "*an IRI statistic is calculated from a single longitudinal profile measured with a road profiler in both the inside and outside wheelpaths of the pavement*." IRI readings were collected prior to, immediately following placement and periodically following construction by Pavement Management with the use of a road profiler. All measurements were reported in increments of $^1/_{10}{}^{th}$ of a mile. Table 9 summarizes the average IRI for each treatment prior to construction. An associated IRI pavement condition scale is provided in Table 10.

In accordance with the "*IRI Pavement Condition Scale*," the entire length of the roadway was considered to be in near mediocre to poor condition prior to construction. It does appear that the pavement designer attempted to account for the various pavement conditions by utilizing a more comprehensive treatment to address areas with higher IRI values.

Figure 10 displays a graphical time series plot of comparative treatment performance, over time, with respect to average IRI values in terms of inches per mile.

Table 9 IRI Summary Prior to Construction.

Treatment Type	Preconstruction IRI (in./mile)
1 ½" AC	134
3" CIR, 1½" AC	298
3" CIR, 3¼" AC	177
4" CIR, 1½" AC	263
4" CIR, 3¼" AC	245
8" RSB, 3¼" AC	355

Table 10 IRI Pavement condition scale.

Condition Term Categories	Interstate	Other
Very Good	<60	<60
Good	60-94	60-94
Fair	95-119	95-170
Mediocre	120-170	171-220
Poor	>170	>220

Immediately following construction, all of the treatments were considered to be in good condition with respect to the Pavement Condition Scale. While many of the treatments remain in good condition following 13 years of service, both the 3 and 4-inch CIR with 1½-inch overlay treatments are in fair condition. Overall, IRI values for each of the six treatment types display a mostly consistent increasing trend over time. At 13 years, the 3-inch CIR with 1½-inch overlay displays the highest IRI values as well as the largest increase in IRI over time. Surprisingly, the standard overlay appears to be outperforming the majority of the other pavement treatments. This is likely due to the condition of these roadway segments prior to construction as the associated IRI values were significantly lower than the IRI values addressed with cold recycled or reclaimed stabilized base treatments.

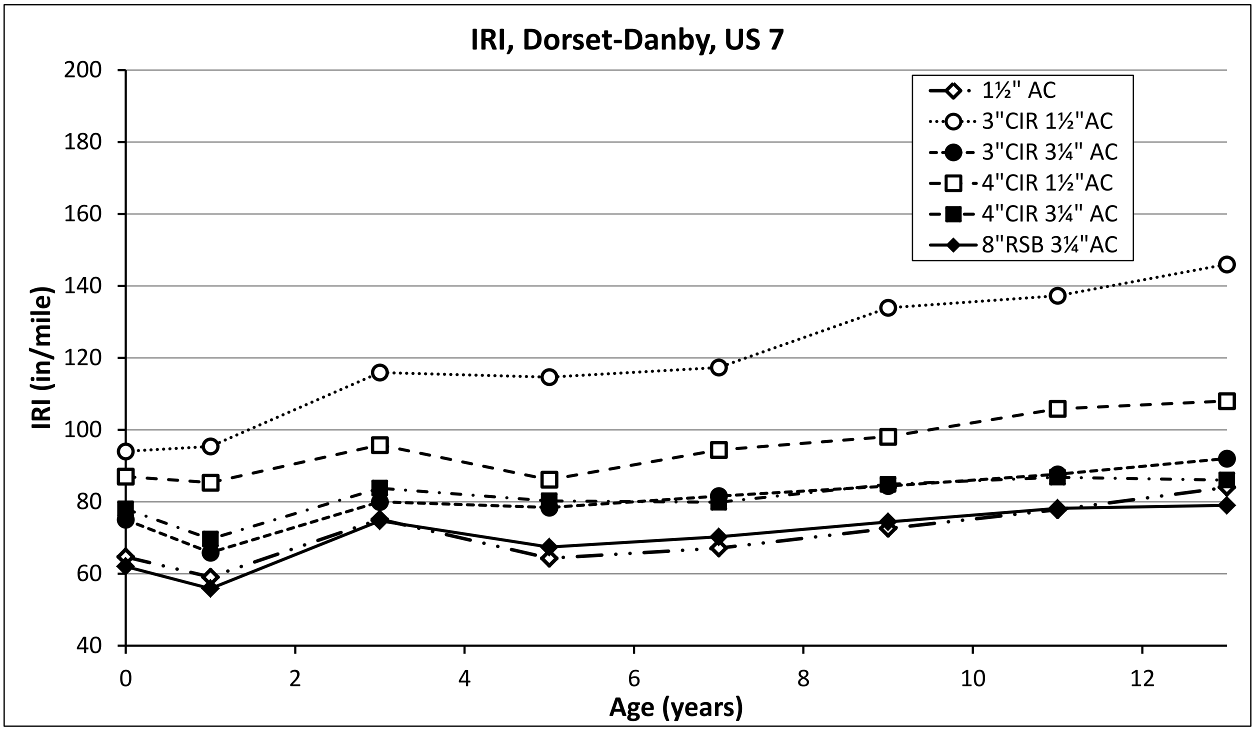

Figure 10 IRI plot for US 7 Dorset/Danby.

CONDITION INDICES

Overall Rating

Given the large number of rehabilitation techniques assessed in this research project along with the various measured pavement distresses, a general rating system was developed to easily compare overall performance of the pavement treatments. This was accomplished by utilizing the current pavement distresses following 13 years of service as a percentage of preconstruction conditions including total cracking, transverse cracking, fatigue cracking, rutting, and IRI as shown in Table 11. A simple pavement condition index was calculated by considering thermal and fatigue cracking, as well as rutting and IRI, with the four weighted equally; the larger the resultant value, the worse the condition of the pavement, with respect to preconstruction values. The final column in the table shows the results of this analysis. A lower value depicts higher performance.

As would be expected, the more comprehensive treatments outperformed the lower cost treatments. In addition, pavement structures with a thicker bituminous overlay displayed fewer distresses over time. Overall, it appears that the 3-inch CIR with 3¼-inch bituminous overlay was the optimum treatment for this roadway segment as it outperformed all of the other treatments and was more cost effective per lane mile as compared to the 4 inch CIR with 3¼-inch overlay and 8-inch RSB with 3¼-inch overlay.

Table 11 Summary of roadway conditions as of year 13, as a percent of preconstruction values. Lower values represent higher performance.

	2015 Per Lane Mile Cost†	Pavement Condition Values as a Percent of Preconstruction					Overall Rating
		Total Cracking	Transverse Cracking	Fatigue Cracking	Rutting	IRI	
1½" AC	$41,128	82	99	53	82	63	**74**
3" CIR, 1½" AC	$62,999	111	65	98	71	49	**71**
3" CIR, 3¼" AC	$110,282	2	4	1	56	52	**28**
4" CIR, 1½" AC	$64,962	145	19	96	48	41	**51**
4" CIR, 3¼" AC	$127,428	10	1	2	136	35	**44**
8" RSB, 3¼" AC	$237,567	64	5	2	120	22	**37**

†See Table 12

Public Benefit

Another relationship was developed to estimate the benefit of the different treatment types with respect to the traffic characteristics (AADT) and roughness (IRI) of the roadway over time, which will be referred to as public benefit. It is a gauge of the perceived ride quality by the public in relation to the amount of daily traffic along this roadway segment. A larger value of the AADT/IRI quantity indicates a higher public benefit. Figure 11 displays the public benefit versus time for the six various treatments, using the IRI values in Table 10 and the AADT values from Table 3.

Interestingly the most and least comprehensive of the six pavement treatments, the RSB and the overlay only treatment, respectively, provide greater public benefit with respect to ride quality. The fact that the 1½-inch overlay was the optimum treatment in this analysis contradicts the previous rating system where the overlay was easily deemed the lowest quality of roadway based on cracking, rutting and IRI. This can be explained by the fact that the previous rating system is based upon comparing current conditions to preconstruction conditions, while the public benefit displays only actual current conditions (per year). Being the least comprehensive (and cheapest) treatment, the overlay only treatment was placed on the sections of roadway that were in the best preconstruction condition, therefore this shows that this treatment type had degraded significantly over its service life as compared to its condition prior to construction (again cracking, rutting, and IRI), while still maintaining a smooth overall ride. The more comprehensive treatments, on the other hand, performed very well in both analysis methods, showing that although the driving public cannot perceive a large difference between the treatments, there is considerable long term structural and monetary benefits to a more extensively rehabilitated roadway.

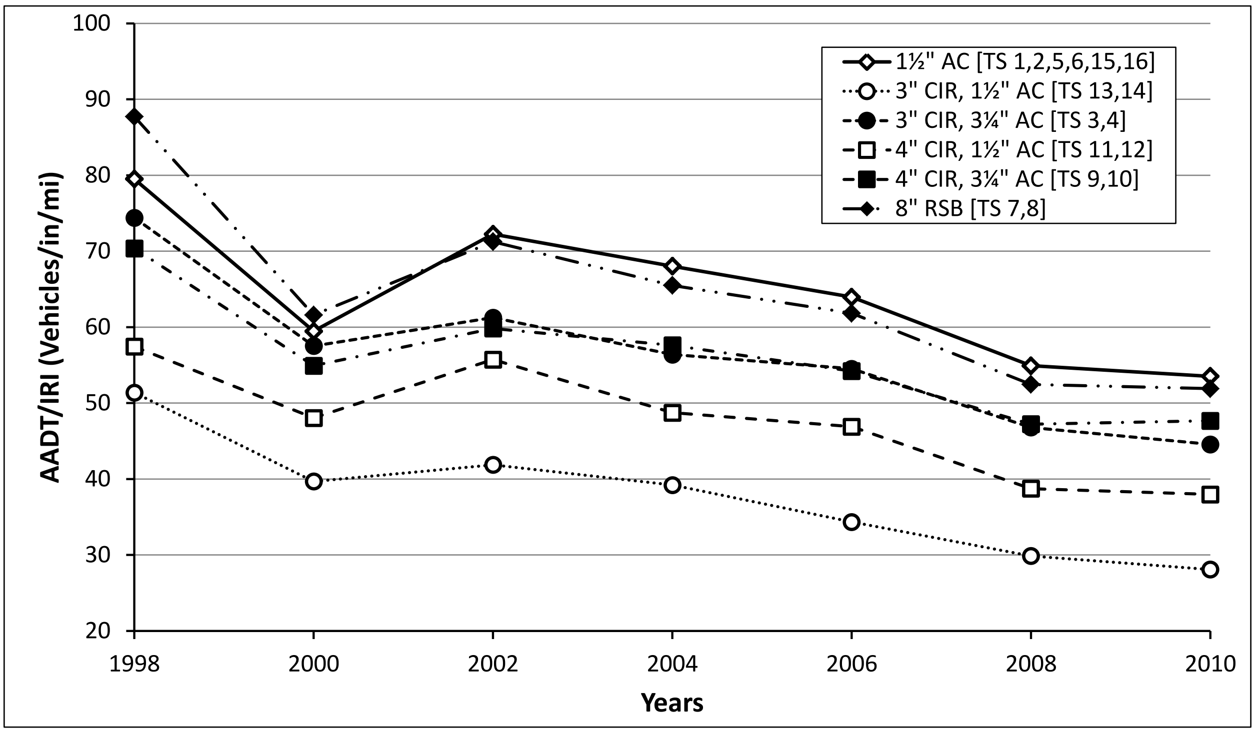

Figure 11 Plot of public benefit (AADT/IRI) versus service years for all treatments.

COST ANALYSIS

The costs associated with Project NH 9514 C/1 included emulsified asphalt at \$10.70/hundred weight (CWT), bituminous concrete pavement at \$25.00/ton, and cold mixed recycled bituminous pavement at \$1.65/yd^2. The costs associated with contract two included emulsified asphalt at \$25/CWT, bituminous concrete pavement at \$36.00/ton, cold planing bituminous pavement at \$2.00/yd^2, and reclaimed stabilized base at \$1.50/yd^2. Please note that in contract two, more than double the amount of emulsified asphalt as originally planned was used due to a revised application range of 0.015-0.03 gal/yd^2. The decision was made to use 0.03 gal/yd^2, reasoning that the overrun would be minor. This was subsequently justified. The total cost per lane mile in 1997 as well as an inflation adjusted cost for 2015 is provided in Table 12.

Table 12 Costs per lane mile as of construction and its associated present day cost.

Treatment	Cost per Lane-Mile (1997)	Cost per Lane-Mile (2015†)
1½" AC	$27,309	$41,128
3" CIR, 1½" AC	$41,831	$62,999
3" CIR, 3¼" AC	$73,227	$110,282
4" CIR, 1½" AC	$43,135	$64,962
4" CIR, 3¼" AC	$84,612	$127,428
8" RSB, 3¼" AC	$157,744	$237,567

†Consumer Price Index, Urban (CPI-U) adjustment from 1997 to 2015

SUMMARY AND RECOMMENDATIONS

This study covered several rehabilitation treatments in mostly homogenous environments with respect to underlying subbase and subgrade soils, traffic volume, existing pavement structure and ambient conditions. In an effort to examine the long-term performance and cost effectiveness of these treatments, the Vermont Agency of Transportation specified the construction of a standard overlay, cold recycled pavement, and reclaimed stabilized base along US Route 7 in the towns of Dorset, Mt. Tabor and Danby in 1996 and 1997. Each of these treatments was intended to address various pavement distresses. For example, CIR pavements were expected to correct functional deficiencies while the RSB process was intended to improve structural deficiencies. Construction costs typically increase with as the treatments become more comprehensive. As such, the overall goal of this project was to determine the optimum treatment for similar sections of roadway with respect to performance and annualized cost.

The Cold In-place Recycling (CIR) treatment to depths of 3 to 4 inches was performed during the fall of 1996. Prior to the onset of the winter season, a bituminous binder or wearing course was applied to each of the four segments in order to protect them from freeze-thaw and tire damage. Specified compaction was not achieved in many locations within the cold recycled pavement. No other problems were reported. The project was completed during the summer of 1997 with the construction of the reclaimed stabilized base and standard overlay segments. A small portion of the roadway was reclaimed to a depth of 8 inches. Compaction was readily achieved. A bituminous binder and wearing course was placed over the reclaimed base. All other remaining segments received a wearing course.

Completed testing and surveillance measures included annual pavement surveys prior to and following construction along with the collection of IRI reading by Pavement Management with the use of a road profiler. Sixteen 100-foot test sections were established throughout the length of the project. Associated pavement surveys included the documentation of cracking and rutting. Cracking was further analyzed for total cracking, fatigue cracking and transverse cracking. Thirteen years following construction, only three of the six pavement composites display measurable amounts of fatigue and transverse cracking including the standard overlay, 3-inch CIR with 1½-inch overlay, and 4-inch CIR with 1½-inch overlay. However, it is impressive to note that current levels of cracking across most test sites have yet to meet or exceed preconstruction conditions. Two of the more comprehensive treatments, 4-inch CIR and 8-inch RSB with 3¼-inch bituminous overlay, displayed the greatest amount of rutting to date as a function of preconstruction conditions. With respect to the IRI and the pavement condition index, the roadway segment was considered to be in mediocre to fair condition prior to rehabilitation. Following construction, the roadway was considered to be in good condition. While many of the treatments remain in good condition following 13 years of service, both the 3 and 4-inch CIR with 1½-inch overlay treatments are in fair condition.

An overall rating system was developed, comparing different aspects of the distresses as compared to preconstruction values. The analysis revealed that the more comprehensive treatments scored a lower overall rating, i.e. were found to be in better current condition with reference to preconstruction condition. The 3-inch CIR treatment outperformed all other treatments (again compared to its own preconstruction condition), while the overlay only treatment was easily the most underperforming; however it was still in good condition with consideration to the IRI Pavement Condition Scale. A public benefit analysis was also conducted, indicating which pavement treatments provide the best-perceived ride to the public. This also showed that, in general, the more comprehensive treatments with the thicker overlays preformed the best with the exception of the overlay only treatment, which was found to provide the best ride. This is due to the fact that the more minimal treatments were applied to the pavement sections that were in the best preconstruction condition; therefore what the public may perceive as being the best ride may not, in fact, be the best treatment to correct and maintain the overall road quality.

IMPLEMENTATION STRATEGY

Overall, a larger timeframe would be required to fully document and determine which of the treatments will ultimately be the best for this section of road. As of the end of data collection, many treatments showed very little cracking, therefore life spans cannot be calculated. More time would need to be afforded to the study in order to represent the benefits of each treatment accurately. Unfortunately, this section of road received a preventative maintenance treatment during the 2011 construction season, therefore ending this research initiative, as originally configured.

The results suggest that CIR with moderate HMA thickness is an excellent treatment for highway on coarse subgrades. Reclamation provided lesser performance and higher costs. Further assessment of network condition and preliminary engineering data needs to be performed before selection of reclamation to confirm the suitability of subbase and subgrade conditions.

APPENDIX A

Table A1 1½" AC

Year	Test Site 2.80 - Dorset (ft/100ft)		Test Site 3.20 - Dorset (ft/100ft)	
	All Transverse Cracking	All Fatigue Cracking	All Transverse Cracking	All Fatigue Cracking
1995	215	563	66	462
1996	Constructed	Constructed	Constructed	Constructed
1997	Constructed	Constructed	Constructed	Constructed
1998	0	0	0	0
1999	0	0	0	0
2000	24	5	0	0
2001	Not Done	Not Done	Not Done	Not Done
2002	59	9	2	57
2003	59	23	10	57
2004	59	33	12	59
2005	Not Done	Not Done	Not Done	Not Done
2006	119	160	32	165
2007	139	179	37	188
2008	174	198	58	222
2009	176	200	66	256
2010	190	214	78	269

Table A2 3" CIR 1¾" Type II Binder 1½" AC

Year	Test Site 5.60 - Dorset (ft/100ft)		Test Site 5.80 - Dorset (ft/100ft)	
	All Transverse Cracking	All Fatigue Cracking	All Transverse Cracking	All Fatigue Cracking
1995	529	150	74	831
1996	Constructed	Constructed	Constructed	Constructed
1997	Constructed	Constructed	Constructed	Constructed
1998	0	0	0	0
1999	0	0	0	0
2000	0	0	0	0
2001	Not Done	Not Done	Not Done	Not Done
2002	0	0	0	0
2003	0	0	0	0
2004	0	0	0	0
2005	Not Done	Not Done	Not Done	Not Done
2006	0	0	0	0
2007	0	0	2	0
2008	1	0	2	0
2009	4	0	2	4
2010	4	0	4	7

Table A3 1½" AC

Year	Test Site 6.20 - Dorset (ft/100ft)		TS 0.00-Dorset/M.Tabor (ft/100ft)	
	All Transverse Cracking	All Fatigue Cracking	All Transverse Cracking	All Fatigue Cracking
1995	109	190	86	361
1996	Constructed	Constructed	Constructed	Constructed
1997	Constructed	Constructed	Constructed	Constructed
1998	0	0	0	0
1999	0	0	0	0
2000	7	0	0	0
2001	Not Done	Not Done	Not Done	Not Done
2002	10	37	4	0
2003	21	37	4	0
2004	21	40	4	0
2005	Not Done	Not Done	Not Done	Not Done
2006	43	94	23	18
2007	43	111	32	33
2008	47	114	38	34
2009	50	121	45	39
2010	56	133	48	52

Table A4 8" RSB 1¾" Type II Binder 1½" AC

Year	TS 0.50 - Mt. Tabor (ft/100ft)		TS 0.72 - Mt. Tabor (ft/100ft)	
	All Transverse Cracking	All Fatigue Cracking	All Transverse Cracking	All Fatigue Cracking
1995	102	249	98	31
1996	Constructed	Constructed	Constructed	Constructed
1997	Constructed	Constructed	Constructed	Constructed
1998	0	0	0	0
1999	0	0	0	0
2000	0	0	0	0
2001	Not Done	Not Done	Not Done	Not Done
2002	0	0	0	0
2003	0	0	0	0
2004	0	0	0	0
2005	Not Done	Not Done	Not Done	Not Done
2006	0	0	0	0
2007	0	0	0	0
2008	0	0	2	0
2009	5	0	4	4
2010	6	0	4	4

Table A5 4" CIR 1¾" Type II Binder 1½" AC

4" CIR 1¾" Type II Binder 1½" AC				
Year	**TS 1.32 - Mt. Tabor (ft/100ft)**		**TS 1.60 - Mt. Tabor (ft/100ft)**	
	All Transverse Cracking	**All Fatigue Cracking**	**All Transverse Cracking**	**All Fatigue Cracking**
1995	97	146	132	339
1996	Constructed	Constructed	Constructed	Constructed
1997	Constructed	Constructed	Constructed	Constructed
1998	0	0	0	0
1999	0	0	0	0
2000	0	0	0	0
2001	Not Done	Not Done	Not Done	Not Done
2002	0	0	0	0
2003	0	0	0	0
2004	0	0	0	0
2005	Not Done	Not Done	Not Done	Not Done
2006	0	0	0	0
2007	0	0	0	0
2008	0	1	2	2
2009	0	1	2	2
2010	0	1	2	2

Table A6 4" CIR 1½" AC

4" CIR 1½" AC				
Year	**TS 2.22 - Mt. Tabor (ft/100ft)**		**TS 2.30 - Mt. Tabor (ft/100ft)**	
	All Transverse Cracking	**All Fatigue Cracking**	**All Transverse Cracking**	**All Fatigue Cracking**
1995	53	163	84	147
1996	Constructed	Constructed	Constructed	Constructed
1997	Constructed	Constructed	Constructed	Constructed
1998	0	0	0	0
1999	0	0	0	0
2000	0	0	0	0
2001	Not Done	Not Done	Not Done	Not Done
2002	0	0	0	33
2003	0	0	0	33
2004	0	0	0	33
2005	Not Done	Not Done	Not Done	Not Done
2006	2	0	0	75
2007	2	83	0	75
2008	Cancelled Due to Alligator Cracking		11	110
2009			13	129
2010			13	148

Table A7 3" CIR 1½" AC

Year	TS 2.50 - Mt. Tabor (ft/100ft)		TS 2.64 - Mt. Tabor (ft/100ft)	
	All Transverse Cracking	All Fatigue Cracking	All Transverse Cracking	All Fatigue Cracking
1995	125	209	150	103
1996	Constructed	Constructed	Constructed	Constructed
1997	Constructed	Constructed	Constructed	Constructed
1998	0	0	0	0
1999	0	0	0	0
2000	0	0	0	0
2001	Not Done	Not Done	Not Done	Not Done
2002	0	0	0	0
2003	0	0	1	0
2004	0	0	9	6
2005	Not Done	Not Done	Not Done	Not Done
2006	5	0	15	19
2007	9	10	35	40
2008	25	39	66	115
2009	26	43	71	137
2010	Patched, cancelled		90	153

Table A8 1½" AC

Year	TS 3.21 - Mt. Tabor (ft/100ft)		TS 0.00 - M. Tabor/Danby (ft/100ft)	
	All Transverse Cracking	All Fatigue Cracking	All Transverse Cracking	All Fatigue Cracking
1995	123	201	6	0
1996	Constructed	Constructed	Constructed	Constructed
1997	Constructed	Constructed	Constructed	Constructed
1998	0	0	0	0
1999	0	0	0	0
2000	16	11	0	0
2001	Not Done	Not Done	Not Done	Not Done
2002	66	55	7	5
2003	66	55	7	12
2004	71	70	9	12
2005	Not Done	Not Done	Not Done	Not Done
2006	122	138	12	12
2007	141	186	12	12
2008	171	239	12	12
2009	178	239	16	12
2010	222	279	16	12

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

Table B1 Summary of the average annual daily traffic (AADT) - US Route 7

Test Section Locations									
Test Section ID	**Begin MM**	**End MM**	**1996 AADT**	**1998 AADT**	**2000 AADT**	**2002 AADT**	**2004 AADT**	**2006 AADT**	**Average AADT**
TS 1	2.800	2.820	4500	4700	4600	4800	4600	4600	4633
TS 2	3.200	3.220	4500	4700	4600	4800	4600	4600	4633
TS 5	6.200	6.220	4500	4700	4600	4800	4600	4600	4633
TS 6	0.000	0.020	4500	4700	4600	4800	4600	4600	4633
TS 15	3.210	3.230	4100	4100	4200	4400	4300	4500	4267
TS 16	0.000	0.020	4100	4100	4200	4400	4300	4500	4267
		Average AADT	**4367**	**4500**	**4467**	**4667**	**4500**	**4567**	**4511**
TS 3	5.600	5.620	4500	4700	4600	4800	4600	4600	4633
TS 4	5.800	5.820	4500	4700	4600	4800	4600	4600	4633
TS 7	0.500	0.520	4500	4700	4600	4800	4600	4600	4633
TS 8	0.720	0.740	4500	4700	4600	4800	4600	4600	4633
TS 9	1.320	1.340	4500	4700	4600	4800	4600	4600	4633
TS 10	1.600	1.620	4500	4700	4600	4800	4600	4600	4633
TS 11	2.220	2.240	4500	4700	4600	4800	4600	4600	4633
TS 12	2.300	2.320	4500	4700	4600	4800	4600	4600	4633
TS 13	2.500	2.520	4500	4700	4600	4800	4600	4600	4633
TS 14	2.640	2.660	4500	4700	4600	4800	4600	4600	4633
		Average AADT	**4500**	**4700**	**4600**	**4800**	**4600**	**4600**	**4633**

APPENDIX C

Table C1 Average Rutting Readings for US 7, Dorset Danby

Year	SB Outer WP		SB Inner WP		NB Inner WP		NB Outer WP	
	1½" AC	3" CIR, 3¼" AC	1½" AC	3" CIR, 3¼" AC	1½" AC	3" CIR, 3¼" AC	1½" AC	3" CIR, 3¼" AC
1996	0.31	0.46	0.26	0.25	0.28	0.31	0.34	0.54
1998	0.00	0.00	0.02	0.06	0.00	0.00	0.01	0.06
1999	0.00	0.00	0.01	0.06	0.01	0.00	0.01	0.08
2000	0.09	0.00	0.13	0.19	0.10	0.02	0.11	0.15
2003	0.14	0.06	0.23	0.25	0.18	0.15	0.17	0.21
2004	0.09	0.04	0.15	0.13	0.12	0.06	0.16	0.13
2006	0.15	0.06	0.26	0.25	0.19	0.19	0.24	0.21
2007	0.18	0.19	0.26	0.31	0.22	0.21	0.29	0.21
2008	0.15	0.10	0.23	0.23	0.17	0.10	0.26	0.18
2009	0.17	0.06	0.31	0.31	0.22	0.15	0.25	0.21
2010	0.23	0.08	0.32	0.38	0.27	0.23	0.30	0.25
Percent of Precon. 1998-09	74%	17%	125%	152%	95%	74%	88%	46%

Table C2 Average Rutting Readings for US 7, Dorset Danby

Year	SB Outer WP		SB Inner WP		NB Inner WP		NB Outer WP	
	8" RSB 3¼" AC	4" CIR 3¼" AC	8" RSB 3¼" AC	4" CIR 3¼" AC	8" RSB 3¼" AC	4" CIR 3¼" AC	8" RSB 3¼" AC	4" CIR 3¼" AC
1996	0.31	0.17	0.38	0.38	0.25	0.25	0.33	0.23
1998	0.04	0.00	0.02	0.02	0.00	0.00	0.02	0.00
1999	0.08	0.00	0.04	0.04	0.04	0.00	0.08	0.00
2000	0.27	0.08	0.13	0.13	0.23	0.17	0.13	0.10
2003	0.27	0.23	0.27	0.27	0.33	0.25	0.19	0.23
2004	0.23	0.19	0.21	0.21	0.29	0.23	0.19	0.21
2006	0.27	0.23	0.29	0.29	0.35	0.27	0.29	0.23
2007	0.35	0.27	0.25	0.25	0.35	0.27	0.33	0.29
2008	0.35	0.29	0.29	0.29	0.31	0.29	0.28	0.33
2009	0.46	0.31	0.29	0.29	0.38	0.31	0.35	0.35
2010	0.48	0.38	0.31	0.42	0.38	0.33	0.33	0.38
Percent of Precon. 1998-09	154%	228%	83%	112%	152%	132%	99%	166%

Table C3 Average Rutting Readings for US 7, Dorset Danby

Year	SB Outer WP		SB Inner WP		NB Inner WP		NB Outer WP	
	4" CIR 1½" AC	3" CIR, 1½" AC	4" CIR 1½" AC	3" CIR, 1½" AC	4" CIR 1½" AC	3" CIR, 1½" AC	4" CIR 1½" AC	3" CIR, 1½" AC
1996	0.33	0.31	0.40	0.31	0.31	0.35	0.42	0.31
1998	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1999	0.00	0.04	0.00	0.04	0.00	0.10	0.00	0.08
2000	0.13	0.08	0.17	0.10	0.19	0.08	0.13	0.13
2003	0.19	0.19	0.19	0.19	0.13	0.21	0.19	0.17
2004	0.13	0.17	0.17	0.21	0.13	0.19	0.15	0.17
2006	0.21	0.17	0.19	0.15	0.04	0.15	0.17	0.10
2007	0.33	0.25	0.25	0.13	0.08	0.19	0.25	0.15
2008	0.29	0.25	0.13	0.21	0.04	0.25	0.25	0.21
2009	0.33	0.29	0.13	0.21	0.00	0.25	0.25	0.17
2010	0.42	0.27	0.21	0.25	0.00	0.25	0.25	0.25
Percent of Precon. 1998-09	126%	86%	53%	80%	0%	71%	60%	80%

APPENDIX D

Table D1 Dorset-Danby Paving Temperatures for late October and November

Date	Town	Average Temperature (^oF)	Minimum Temperature (^oF)
10/23/1996	Mt. Tabor	48	37
10/24/1996	Mt. Tabor	50	44
10/25/1996	Mt. Tabor	52	48
10/26/1996	Dorset	46	32
10/30/1996	Dorset	43	35
10/31/1996	Dorset	43	35
11/6/1996	Mt. Tabor	44	39
11/8/1996	Mt. Tabor	66	62
11/11/1996	Mt. Tabor	32	28
11/26/1996	Mt. Tabor	26	23

Table D2 Low temperatures for each year

Year	Low Temp
1	-13
2	-14
3	-11
5	-2
6	-20
7	-16
9	-5
10	-14

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