Evaluation of Concrete Bridge Rail Cracking
Initial Report

October 2009

Report 2009 - 5

State of Vermont
Agency of Transportation
Materials and Research Section

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Date: October 8, 2009

1. Report No. 2009-5	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle **Evaluation of Concrete Bridge Rail Cracking Initial Report**		5. Report Date October 2009 6. Performing Organization Code
7. Author(s) Jason P. Tremblay M.S., Jennifer M. V. Fitch P.E.		8. Performing Organization Report No. 2009-5
9. Performing Organization Name and Address Vermont Agency of Transportation Materials and Research Section National Life Building Drawer 33 Montpelier, VT 05633-5001		10. Work Unit No. 11. Contract or Grant No.
12. Sponsoring Agency Name and Address Federal Highway Administration Division Office Federal Building Montpelier, VT 05602		13. Type of Report and Period Covered Initial (2008-2009) 14. Sponsoring Agency Code
15. Supplementary Notes		

16. Abstract

Cracking of newly cast concrete bridge rails has been an ongoing problem throughout the State of Vermont, one that has been observed to be severe and rapidly occurring upon curing in some cases. This study reports on the current status with regards to cracking and condition of eleven concrete bridge rails, eight previously cast since 2001 and three which were monitored from casting through their first year of service.

Through field observations it was determined that all bridge rails showed signs of cracking. For rails with windows (cut-outs), the crack density was around 0.8 cracks per linear foot, while for solid section rails it was around 0.5 cracks per foot. Through monitoring of the three rails from casting, it is apparent that the cracking develops very quickly and increases rapidly during the first months.

Corollary statistics show some general trends with respect to geometric, functional, and materials data. It is apparent that there is a widespread problem, one that needs to be alleviated before resources are poured into a currently inconsistent product.

17. Key Words	18. Distribution Statement
Concrete bridge rails Shrinkage cracking	No restrictions

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. Pages	22. Price
Unclassified	Unclassified	49	

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Executive Summary

Cracking of newly cast concrete bridge rails has been an ongoing problem throughout the State of Vermont, one that has been observed to be severe and rapidly occurring upon curing in some cases. Cracking in concrete allows for the penetration of chlorides and other corrosives resulting in deterioration of reinforcing. Additionally, moisture may also penetrate more readily resulting in accelerated damage from freeze thaw cycles in the form of scaling and spalling. These stressors lead to decreased strength and safety of the rails, increased maintenance costs, a reduction in the overall aesthetics of the structures and decrease in public confidence and support.

This study documents the current condition of eleven concrete bridge rails with special emphasis of observed surface cracks of which eight were preexisting prior to the onset of this report and three were cast over the study period. This allowed for continuous monitoring of construction practices including pouring, consolidation, curing and finishing, and initial performance.

One site visit was conducted at each of the previously constructed rails in an effort to document any observed cracking along the concrete bridge rails. Multiple site visits were conducted to newly cast rails over the first year of surface in an effort to examine the onset, rate and extent of cracking. Overall, cracking was evident along all bridge rails within the sample population regardless of the various parameters including the mix design, curing methods, design, length, age, and traffic volume. For rails with windows (cut-outs), the crack density was found to be 0.75 cracks per linear foot (or one crack per 1.25 feet), while for solid section rails it was roughly 0.46 cracks per foot (or one crack per 2 feet), using a normalization technique for windowed rails, see Section 4. Observations drawn from newly cast rails indicate that cracks develop rapidly following construction and continue to develop through the first year of service.

Corollary statistics display some general trends with respect to bridge geometry, functional classification, and concrete mix design. Orientation was found to be an exploratory variable with rails facing north displaying less cracking than southerly facing rails. Bridge length and number of main spans both showed a small positive correlation to cracking (the longer the bridge and/or the more spans the greater the cracking), while traffic considerations had a negligible effect. Regression analysis was performed in order to identify which admixtures or cement contents were found to impact the amount of cracking within the sample population. Two admixtures found to influence cracking were the Eclipse Plus shrinkage control admixture, resulting in a slight decrease in cracking, and Daratard 17 initial set retarder, which was found to minimally increase cracking. All other admixtures and cement contents analyzed were found to have a neutral affect. A larger population would need to be tested in order to verify these claims as being statistically significant.

Cracking in these concrete rails may be attributed to the recent use of high performance (HP) portland cement concrete, a mixture specified to meet special combinations of performance and uniformity. For example, in accordance with the Agency's

specifications, HP classes AA and A are required to obtain a minimum 28 day compressive strength of 30 MPa. In addition, when 90% of the 28-desgin strength is obtained at 14 days, 28 day testing may be omitted. As such, associated materials testing have revealed excess concrete strengths at 14 days. While high strength concretes are advantageous for structural applications, they also are prone to cracking. Therefore, it is hypothesized that observed cracking is most likely due to higher strength concretes, shrinkage cracking during the curing process and dynamic live load distribution along the rails. This is supported by cracking observed prior to the onset of traffic and the formation of cracks during service.

Both contractors and Agency personnel alike indicate that the concrete rails are much more time, labor, and cost intensive to produce than installing other alternatives such as steel rails onto a bridge. With this in mind as well as the fact that there is a fair amount of premature cracking, it is apparent that there is a widespread problem with respect to our current concrete bridge rail practices; one that needs to be addressed and alleviated.

1. Introduction

Cracking of newly cast concrete bridge rails has been an ongoing problem throughout the State of Vermont, one that has been observed to be severe and rapidly occurring upon curing in some cases. Cracking in concrete allows for the penetration of chlorides and other corrosives resulting in deterioration of reinforcing. Additionally, moisture may also penetrate more readily resulting in accelerated damage from freeze thaw cycles in the form of scaling and spalling. These stressors lead to decreased strength and safety of the rails, increased maintenance costs, a reduction in the overall aesthetics of the structures, and decrease in public confidence and support.

Generally, concrete is comprised of four basic elements, Portland cement, a fine aggregate, a coarse aggregate, and water. In most cases, pozzolans and admixtures are also incorporated into the mix, for specific characteristics such as workability or air entrainment. It should be noted that “high performance” (HP), mixes were introduced in the 1990’s in an effort to preclude Alkali-Silica-Reactivity (ASR) distress in concrete. HP mixes substitute fly ash and microsilica for a portion of the cement. Other beneficial traits of HP mixes are their high strengths and low permeability. The State of Vermont allows several different options for the proportioning of high performance concrete compositions. Once thoroughly mixed, this matrix is poured into forms, around reinforcing steel, and then vibrated until properly consolidated. Sections that will be visible are usually finished to proper contour and elevation. Finally, the cast concrete cures for strength gain, improved durability and enhanced resistance to wear. Curing is essential for optimum performance and is typically accomplished through a wet curing process where the concrete is surface flooded, ponded or mist sprayed as well as covered to retain water and reduce wicking. Curing times will vary depending on several mix design and weather parameters. Ambient air temperature is another important parameter as higher temperatures increase the initial rate of strength gain.

Aesthetic bridge rails, patterned after a Texas Rail, have been implemented on more and more bridge designs over the past half dozen years. During the earlier part of the 20th century, concrete rails were used abundantly throughout the State of Vermont, particularly in the 1920s and decades that followed. Patterns for the rail included spindles, windows and recessed panels. Many of these structures have reached the end of their service lives and are being replaced. As such, towns and historic societies are requesting comparable concrete bridge rail replacements.

Texas rails are intricate in visual design and pose additional challenges during construction in comparison to other bridge rail alternatives, such as steel rails, requiring a considerable amount of time, effort, and money to produce. Steel members can simply be brought on site and bolted into place in a day or so, while methods of concrete placement require several weeks from pouring to finishing. A concrete rail must be poured into specially made (normally onsite) forms specific to each bridge. These forms are normally comprised of plywood with some sort of filler material, such as polystyrene, used to block out the "window" openings. The forms are placed around the reinforcing steel. Reinforcing is needed as concrete is only strong in compression; therefore reinforcing steel is needed to provide tensile strength. Once the concrete is poured and consolidated into the forms it must be kept hydrated and, in cold weather, sometimes heated (depending on the ambient temperatures) for proper curing. It is often difficult to consolidate the concrete at the bottom of the forms during the vibrating process. Air pockets often migrate to the inside face of the forms becoming a permanent feature of the rail once the forms are removed. To combat these problems, a smaller maximum aggregate size of 3/8" is specified in the mix design as compared to ¾" for bridge decks.

In accordance with the "2006 Standard Specifications for Construction," concrete superstructure elements must be cured for ten days. With concrete bridge rails, this is generally accomplished by leaving the concrete inside the forms for roughly two to three days until the concrete has gained sufficient strength to retain shape and hardness to resist surface damage generated by the removal of the forms. Once the forms are removed, continued curing is specified. Two options are available. A chemical membrane may be applied to any vertical exposed surfaces, referred to as a curing compound. Wet cures, employing the use of water, are also attempted; however it is often difficult to maintain a wet cure on vertical surfaces or the underside of horizontal surfaces. Once properly cured the forms are removed and the surfaces of the concrete must be finished, which usually entails 'rubbing' for surface texture; this process is very time intensive. Finally, a coat of silane waterproofing is often applied to the concrete in an attempt to protect it from the elements. This entire process can require a month of elapsed time to complete for structures around 150 feet in length or more. In informal discussions with Agency personnel and contractors associated with the production of the rails, concrete rails are not preferred..

While there are many causes for cracking in concrete, shrinkage cracking is the most common. As a standard practice, concrete is mixed with more water than is exactly needed to hydrate the cement. As the cement cures, much of the remaining water is rapidly used up during the hydration process. Water on and near the surface evaporates causing the concrete to shrink. Shrinkage cracking of the concrete is restrained by the steel reinforcing causing tensile stresses to develop in the hardened concrete ultimately resulting in cracking. Cracking may also be attributed to the use of HP mixes. In accordance with the Agency's specifications, HP classes AA and A are required to obtain a minimum 28 day compressive strength of 30 MPa. In addition, when 90% of the 28-desgin strength is obtained at 14 days, 28 day testing may be omitted. As such, associated materials testing have revealed excess concrete strengths at 14 days. While high strength concretes are advantageous for structural applications, they also are prone to cracking. Finally, assuming composite action of the bridge decks and rails, live loads may produce cracking over time. Therefore observed cracking may be due to shrinkage cracking, higher strength concretes and dynamic live load distribution along the rails.

1.1. Objectives

The objective of this research initiative was to record the occurrence of cracking in Vermont's newly cast concrete bridge rails. In some instances, rails were cast during this research initiative allowing for an examination of the onset and rate of cracking following the curing process. This was accomplished by documenting current practices with respect to mix designs, casting and curing methods. Photographs and written documentation of the current state of recently cast rails were collected. Particular attention focused on the phenomena of cracking including the attributes of size, pattern, placement, and occurrence (temporal appearance). Several other parameters were assessed including mix designs with respect to water/cement ratio, air content, admixtures, silica fume, pozzolans, slag and aggregate gradation, construction methods including pouring, cast and curing methods, and rail design.

Methodologies used to perform the study included an in depth literature search, a survey of surrounding states and provinces to determine if similar problems exist throughout the region, a complete document review pertaining to selected structures, field data collection, and, finally, statistical analysis in order to examine possible correlations between the parameters described above and occurrence of cracking.

2. Presentation of Structures

Bridge structures observed within the scope of this project were selected by VTrans Structures personnel. This is not to be considered an exhaustive list, as there may be others that were not referred to the investigators. All bridges were built beginning in 2001 and therefore can be considered 'new' bridges. Each structure in the study employed the use of "high performance", or HP, mixes. HP mixes were introduced in the 1990's in an effort to preclude Alkali- Silica-Reactivity distress in concrete, where a portion of the cement is substituted with fly ash, microsilica, and/or ground granulated blastfurnace slag (GGBS). Additional traits of HP mixes are their high strengths and low permeability. The State of Vermont allows several different options for the proportioning of high performance concrete compositions. While bridges included within this study share the trait of concrete bridge rails cast with HP mixes, many aspects vary widely between bridges such as: length, bridge type, number of spans, mix design, orientation, and rail type.

All the selected bridge rails fall within one of two categories. The first category contains the bridge rails that were cast prior to the commencement of this study. These structures were all constructed between the 2001 and 2006 construction seasons. Since they were not examined during construction, it is impossible to determine how cracking, if present, progressed following the curing process. Only the current condition of the rails was documented. The second category of bridge rails encompasses those that were cast following the commencement of this study. Unlike the preexisting rails, placement techniques were observed during construction in addition to periodic examination to determine the onset and rate of cracking. This group was used to more accurately determine when the onset of cracking occurred and how it progressed throughout the given timeframe.

Tables 1 and 2 below present an overview of the bridges selected within each of the categories, along with other pertinent data comparing the different structures. A more detailed description of each bridge is provided in Sections 2.1 and 2.2. For rail types, the abbreviations “Tex” refers to Texas bridge rails containing windows, while “Mod” stands for modified and refers to solid rail sections of a few varying types.

Table 1. Previously constructed concrete bridge rails.

Town	Route	Bridge No.	Date of Pour	Length	Rail Type
Bethel	VT 12	31	Fall 2006	183’	Mod
Chester	VT 103	12	Fall 2004	112’	Tex
S. Burlington	Lime Kiln Rd.	6	Fall 2006	298’	Tex
Corinth	FAS 193	10C	Fall 2004	72’	Mod
Lyndon	Alt VT 122	2	Fall 2004	172’	Tex
Morristown	VT 100	213	Summer 2002	148’	Mod
Tunbridge	VT 110	4	Winter 2006-07	184’	Tex
Underhill	FAS 233	9	Fall 2001	60’	Tex

Table 2. Bridge rails constructed and monitored during the period of this project.

Town	Route	Bridge No.	Rails Poured	Length	Rail Type
Barton	C2003	61	Fall 2007	114	Tex
Randolph	VT 12	42	Fall 2007	204	Tex
Warren	FAS 188	7	Fall 2007	64	Mod

2.1. Previously Constructed Structures

As presented in Table 1, eight previously cast bridge structures were incorporated into this examination. Each bridge is described in detail in the following sections, with an associated table of design aspects and photograph of each structure.

2.1.1. Bethel

The bridge in Bethel is comprised of a 'composite' bridge rail, meaning it consists of both concrete and steel portions. The base of the rails is comprised of solid sections of concrete with an exposed aggregate finish. In addition, steel guard posts and rails have been bolted on top of the concrete base as shown in Figure 1. Table 3 displays some of the various other characteristics of the bridge structure.

Table 3. Pertinent design aspects of the Bethel bridge.

Location	Located On	VT 12
	Bridge Number	31
	Feature Crossed	3rd Br. White R. & RR
Age & Service	Year Built	2006
	Year Reconstructed	--
	AADT	6300
	Truck ADT	3%
Geometry	Length of Max Span (ft)	141
	Structure Length (ft)	183
	Deck Width (ft)	37.4
	Skew	0
	Orientation from North	43°
Structure Type & Materials	Bridge Type	Welded Plate Girder
	Number of Approach Spans	1
	Number of Main Spans	1
	Kind of Material	Steel
Condition	Federal Sufficiency Rating	72.7 (10/15/07)

Rails were poured on four days from September 18 through September 28, 2006, with rubbing and finishing finalized on October 5, 2006. In all cases, forms were stripped off the day following casting and sprayed with a pressure washer in an attempt to expose the underlying aggregate for a 'pebble' finish along the exposed surfaces. This did not work as well as planned, as some panels contained an adequate 'pebble' finish while others retained a flat appearance. According to onsite personnel, this discrepancy was preliminarily due to insufficient construction practices as greater sufficiency with form release agents were developed during the finishing process.

Figure 1. Overview of the Bethel bridge.

2.1.2. Chester

The bridge in Chester contains a standard Texas style bridge rail, consisting of windows through the sections. Table 4 summarizes other important factors of the bridge design.

Table 4. Pertinent design aspects of the Chester bridge.

Location	Located On	VT 103
	Bridge Number	12
	Feature Crossed	Williams River
Age & Service	Year Built	2004
	Year Reconstructed	--
	AADT	4420
	Truck ADT	12%
Geometry	Length of Max Span (ft)	107
	Structure Length (ft)	112
	Deck Width (ft)	41.8
	Skew	30°
	Orientation from North	82°
Structure Type & Materials	Bridge Type	Curved Wld Plt Girder
	Number of Approach Spans	0
	Number of Main Spans	1
	Kind of Material	Steel
Condition	Federal Sufficiency Rating	93.2 (4/19/07)

Casting of the rails took place on September 10th and 21st of 2004, with forms removed within two weeks of these dates. Upon form removal, rails were rubbed periodically and finally coated with silane for waterproofing on November 4, 2004. Figure 2 shows the finished product.

Figure 2. Overview of the Chester bridge.

2.1.3. Colchester - South Burlington

The Colchester – South Burlington bridge is also known as the Lime Kiln Bridge. It was listed by "Roads and Bridges" magazine as number two on their annual list of the top 10 bridge projects in North America for 2006. The bridge is unusual for Vermont as it is an open span concrete arch structure, as shown in Figure 3. The rails are Texas style with windows. This is the longest bridge in this study.

Rails were cast during 6 pours, with east and west sections poured between September 8 to 12 and October 3 to 10, 2006, respectively. Form removal and rubbing occurred within two weeks of pouring. Silane was applied to the east sections on September 22, and to the west side on November 1. A lookout pad was also constructed as part of this project with the same rail as the bridge. These rails were poured well after the bridge rails, on June 1 and 5, 2007. The main difference between the bridge rails and the lookout rail is that the lookout rail is cast on grade (i.e. no apparent loading stresses), while the bridge rails are subject to dead and live loads.

Table 5. Pertinent design aspects of the Colchester-South Burlington bridge.

Location	Located On	Lime Kiln Rd.
	Bridge Number	6
	Feature Crossed	Winooski River & RR
Age & Service	Year Built	2006
	Year Reconstructed	--
	AADT	7800
	Truck ADT	3%
Geometry	Length of Max Span (ft)	231
	Structure Length (ft)	298
	Deck Width (ft)	41.7
	Skew	0
	Orientation from North	78°
Structure Type & Materials	Bridge Type	Open Span Conc. Arch
	Number of Approach Spans	2
	Number of Main Spans	1
	Kind of Material	Concrete
Condition	Federal Sufficiency Rating	90.6 (10/11/06)

Figure 3. Overview of the Colchester - South Burlington bridge.

2.1.4. Corinth

Corinth is the first bridge which is listed as being a rehabilitated bridge in this list. It was originally built in 1925. The construction consisted of the rehabilitation of the existing abutments and pier with an entirely new superstructure. Since the steel superstructure of the bridge is completely new, it is feasible to assume that the bridge rails will act as if they were on an entirely new structure. The bridge rail design for this bridge consists of solid sections with a reduced thickness panel.

Table 6. Pertinent design aspects of the Corinth bridge.

Location	Located On	TR 1, FAS 193
	Bridge Number	10C
	Feature Crossed	Waits River
Age & Service	Year Built	1925
	Year Reconstructed	2004
	AADT	1700
	Truck ADT	4%
Geometry	Length of Max Span (ft)	35
	Structure Length (ft)	72
	Deck Width (ft)	26.8
	Skew	0
	Orientation from North	63°
Structure Type & Materials	Bridge Type	2 Spn. Cont. R. C. Slab
	Number of Approach Spans	0
	Number of Main Spans	2
	Kind of Material	Concrete
Condition	Federal Sufficiency Rating	77.5 (7/12/06)

Pouring of the rails occurred on September 23 and 28 of 2004. In both cases form removal and rubbing began the day after casting and finished three days later. Silane was applied to the rails on October 27, 2004.

Figure 4. Overview of the Corinth bridge.

2.1.5. Lyndon

As with the previous Corinth bridge, the Lyndon bridge was rehabilitated with a new deck system. In 2004 the entire deck and rail structures were replaced with new concrete and reinforcing steel. This bridge is unique amongst the sample bridges as it is the only one consisting of three simple spans, which would produce a different stress profile across the structure. Rail design consists of partial window cut, i.e. a deep stencil on the surface.

Table 7. Pertinent design aspects of the Lyndon bridge.

Location	Located On	Alt VT 122
	Bridge Number	2
	Feature Crossed	Passumpsic River
Age & Service	Year Built	1961
	Year Reconstructed	2004
	AADT	3030
	Truck ADT	5%
Geometry	Length of Max Span (ft)	82
	Structure Length (ft)	172
	Deck Width (ft)	38.8
	Skew	0
	Orientation from North	45°
Structure Type & Materials	Bridge Type	3 Span Rolled Beam
	Number of Approach Spans	0
	Number of Main Spans	3
	Kind of Material	Steel
Condition	Federal Sufficiency Rating	85.7 (11/07/07)

Figure 5. Overview of the Lyndon bridge.

Rails for the Lyndon bridge were cast on eight different days, beginning on November 1, 2004 and through on December 8. Rails sections were wrapped with plastic and heated to assist in curing. Exact dates of form removal, rubbing, and silane application were not available.

2.1.6. Morristown

The Morristown bridge is a unique Vermont structure due to the fact that its design and deck construction integrates the use of glass fiber reinforced polymer reinforcement in concert with an associated research project; rail reinforcement, however, was conventional According to the last bridge inspection report on July 18, 2007, "the overall condition of this bridge is satisfactory to good except for the heavy vertical sag of the deck surface and negative camber on all girders." It is important to note that the bridge is in a sag vertical curve, with respect to the roadway. The bridge was intended to match this curve, but with the redesign of the reinforcement, the dead load deflection was underestimated. As such, the bridge deflected under its dead load more than was predicted. This condition may have had some effect on the condition of the bridge rails due to varying stress distribution, however it may not be likely, as results from this bridge closely resembled those for other bridges as will be discussed in later sections.

Table 8. Pertinent design aspects of the Morristown bridge.

Location	Located On	VT 100
	Bridge Number	213
	Feature Crossed	Ryder Brook
Age & Service	Year Built	2002
	Year Reconstructed	--
	ADT	8200
	Truck ADT	9%
Geometry	Length of Max Span (ft)	144
	Structure Length (ft)	148
	Deck Width (ft)	37.0
	Skew	0
	Orientation from North	72°
Structure Type & Materials	Bridge Type	Welded Plate Girder
	Number of Approach Spans	0
	Number of Main Spans	1
	Kind of Material	Steel
Condition	Federal Sufficiency Rating	73.2 (7/18/07)

Casting of these bridge rails took place beginning on June 27, 2002 and continued through July 19, for a total of eight pour dates. No dates or comments were found referencing form removal or finishing of the rails. Rail design for this bridge consisted of solid sections with different surface finishing on the inner rectangular panels than was present on the outer areas, for aesthetic purposes.

Figure 6. Overview of the Morristown bridge.

2.1.7. Tunbridge

This bridge in Tunbridge was the first bridge brought to attention by the Structures Section as having extensive cracking. The rails were finished in early 2007 and during the following summer, extensive cracking was noted by VTrans personnel, which was the catalyst for this research initiative.

Table 9. Pertinent design aspects of the Tunbridge bridge.

Location	Located On	VT 110
	Bridge Number	4
	Feature Crossed	1st Branch White River
Age & Service	Year Built	2007
	Year Reconstructed	--
	AADT	1900
	Truck ADT	2%
Geometry	Length of Max Span (ft)	92
	Structure Length (ft)	184
	Deck Width (ft)	35.7
	Skew	45°
	Orientation from North	40°
Structure Type & Materials	Bridge Type	2 Span Cont Weld Gird
	Number of Approach Spans	0
	Number of Main Spans	2
	Kind of Material	Steel Continuous
Condition	Federal Sufficiency Rating	77.6 (9/05/07)

Rail pouring commenced on November 17, 2006 and finished on January 8, 2007. There were a total of 10 pour dates spread fairly evenly across this timeframe. Rail sections were cast in a leapfrog pattern, i.e. alternating sections and then filled in between afterwards. Forms remained on the concrete anywhere from 2 to 12 days depending on the conditions and were rubbed soon thereafter. Silane was applied within one month of casting in all cases. Rails for this bridge include gothic style windows with a top-mounted steel rail and lamp posts anchored to alternating pilasters.

Figure 7. Overview of the Tunbridge bridge.

2.1.8. Underhill

The Underhill bridge is the oldest structure that was observed in this study. It was built in 2001, and was approximately 82 months old during the time of inspection. This bridge also used slightly different admixtures than the majority of the other bridges in this study, as discussed in Section 4.2.2.

Underhill rails were poured on two dates, October 3, 2001 for the upstream portion of the rail and October 8 for the downstream portion. Forms were stripped and rubbing began on the following day for the upstream portion. Information regarding the removal of forms or rubbing was not available for the downstream side. Rail design is Texas rail with windows.

Table 10. Pertinent design aspects of the Underhill bridge.

Location	Located On	TR 1, FAS 233
	Bridge Number	9
	Feature Crossed	Browns River
Age & Service	Year Built	2001
	Year Reconstructed	--
	AADT	2730
	Truck ADT	6%
Geometry	Length of Max Span (ft)	58
	Structure Length (ft)	86
	Deck Width (ft)	29.0
	Skew	25°
	Orientation from North	40°
Structure Type & Materials	Bridge Type	PS/PT Conc Box Beam
	Number of Approach Spans	0
	Number of Main Spans	1
	Kind of Material	Prestressed Concrete
Condition	Federal Sufficiency Rating	75.7 (9/12/06)

Figure 8. Overview of the Underhill bridge.

2.2. Structures Constructed during the Study

As presented in Table 2, three structures were selected because the concrete bridge rails were scheduled to be poured soon after the commencement of this project. These three rails were observed during construction and the hardened concrete was monitored on a regular basis thereafter for the first year. Each of the three bridges are discussed in detail in the following sections.

2.2.1. Barton

One method of trying to mitigate the shrinkage cracking during the curing process is through the use of a shrinkage compensating admixture. The rails along the Barton deck incorporated such an admixture under a Category II work plan (WP-2007-R-2), which details the execution of the product evaluation. By using this type of admixture, it was anticipated that the rails would not crack, or that the severity of the cracking would be limited. Table 11 below summarizes important aspects of the bridge, while Table 12 shows the contents of concrete mix utilized for the upstream concrete bridge rail on the day of observed placement.

Table 11. Pertinent design aspects of the Barton bridge.

Location	Located On	C2003
	Bridge Number	61
	Feature Crossed	Willoughby River
Age & Service	Year Built	2007
	Year Reconstructed	--
	AADT	740
	Truck ADT	20%
Geometry	Length of Max Span (ft)	110
	Structure Length (ft)	114
	Deck Width (ft)	31.6
	Skew	0
	Orientation from North	61°
Structure Type & Materials	Bridge Type	Welded Plate Girder
	Number of Approach Spans	0
	Number of Main Spans	1
	Kind of Material	Steel
Condition	Federal Sufficiency Rating	88.8 (12/06/07)

Table 12. Concrete mix constituents for upstream Barton bridge rail.

Material	Dosage	Purpose
Adva 100	7.5 oz/y^3	Water Reducer
Daratard 17	3.2 oz/y^3	Extends Setting Time
Darex II	6.3 oz/y^3	Air Entrainer
Eclipse Plus	192 oz/y^3	Shrinkage Control
Flyash	122 lb/y^3	
Silica Fume	40 lb/y^3	
Type II Cement	571 lb/y^3	

Rails for the Barton bridge were poured on two days. The downstream (northbound) side was cast on November 1, 2007, while the upstream (southbound) was cast on November 5. In both instances forms were removed 11 days after pouring, at which time rubbing of the rails commenced. Silane was applied to both rails on November 26. Rail design for this bridge consisted of Texas style rails with windows.

Figure 9. Overview of the Barton bridge.

2.2.2. Randolph

The bridge in Randolph is the second longest structure in this study and was constructed during the 2007 construction season. The rails are Texas rails with very large gothic style windows.

Table 13. Pertinent design aspects of the Randolph bridge.

Location	Located On	VT 12
	Bridge Number	42
	Feature Crossed	3rd Br. White River
Age & Service	Year Built	2007
	Year Reconstructed	--
	AADT	16680
	Truck ADT	6%
Geometry	Length of Max Span (ft)	100
	Structure Length (ft)	204
	Deck Width (ft)	42.3
	Skew	9°
	Orientation from North	85°
Structure Type & Materials	Bridge Type	2 Span Cont Curv Gird
	Number of Approach Spans	0
	Number of Main Spans	2
	Kind of Material	Steel Continuous
Condition	Federal Sufficiency Rating	70.2 (12/07/07)

Downstream portions of the rail were poured during six days, from October 3 to 10, 2007 and the upstream were completed during an eight day period, from October 11 to 22. It was noted by the resident engineer that air contents of the concrete deliveries decreased steadily during the pours for the downstream rail on a daily basis, but were still within specification. For one section, “Section 7,” of the upstream rail, on the other hand, the air content was below specification. Section 7, from station 17+04 to 17+38 Lt, was poured on October 18. Forms were removed in all cases after only one day so that rubbing could begin in order to fill in any existing voids and smooth the finish of the rails. The rails were then kept wet by a hose and burlap for 10 days.

Figure 10. Overview of the Randolph bridge.

2.2.3. Warren

The Warren bridge is the shortest bridge in this study at 64 feet long, with one of the lowest ADTs of 1200. This is the last of the three rehabilitated structures in the study, originally constructed in 1947. While the substructure components were reused, the entire superstructure of the bridge was replaced. Once the concrete was cured, a steel tube railing was inserted along the top of the concrete rail portion, fastened by plates bolted into the concrete.

Since this was a rehabilitated bridge it remained open to one lane of traffic during construction.. The downstream portion of the rail was poured on September 26 and 28, 2007, with traffic remaining on the upstream side during this construction. The upstream was poured over a month later on November 8, during which time traffic was maintained on the downstream side of the bridge. Forms were removed for all sections of rail exactly seven days post pour with the surfaces kept wet with a hose and burlap. For the downstream rail, the five sections were poured in a leapfrog pattern with sections 1, 3, and 5 done the first day and sections 2 and 4 done on the second day. All sections of the upstream side were poured the same day.

Table 14. Pertinent design aspects of the Warren bridge.

Location	Located On	Brook Road
	Bridge Number	7
	Feature Crossed	Freeman's Brook
Age & Service	Year Built	1947
	Year Reconstructed	2007
	ADT	1200
	Truck ADT	4%
Geometry	Length of Max Span (ft)	62
	Structure Length (ft)	64
	Deck Width (ft)	27.0
	Skew	30°
	Orientation from North	34°
Structure Type & Materials	Bridge Type	Steel Beam
	Number of Approach Spans	0
	Number of Main Spans	1
	Kind of Material	Steel
Condition	Federal Sufficiency Rating	79.6 (1/20/08)

Figure 11. Overview of the Warren bridge.

3. DATA COLLECTION

Data collected in conjunction with this study included the number, location, type, and orientation of cracks within the concrete along with the approximate lengths and widths of the cracks. The method used to record this data was a 'crack diagram'. Crack diagrams are scale drawings of each concrete bridge rail associated with each bridge incorporated into the study. They were used to record the location and severity of all observable surface cracks.

Crack diagrams were developed for each bridge in the study. Prior to visiting each (previously constructed) bridge, construction plans, inspection pictures, and Visidata road video software were utilized to determine the geometry of each bridge rail. Geometrical factors important to these diagrams were number of sections, locations of pilasters, number of windows, type of rail (Texas or solid), and rail abutment types. Schematic diagrams had to be large enough to accurately draw on; therefore some longer bridges required a considerable number of pages for their diagrams. Two such diagrams are provided in Appendix A. Included are diagrams from the Lime Kiln bridge (with windows) and the Morristown bridge (solid section). Blank and completed forms are supplied for both.

The approximate width of each crack was also noted on the crack diagrams. Crack widths were determined via the use of a common ‘crack comparator card’ donated by the CTL Group as shown in Figure 12. This transparent card is approximately the size of a credit card and contains reference lines for measuring crack widths in either mm or inches. Inches were utilized throughout this investigation for consistency and to reduce user error. Widths were estimated, by use of the cards, at a location along the crack representative of the average crack width. Therefore, the maximum width of the crack may not have been recorded.

Figure 12. Crack comparator card used to estimate crack widths.

One site visit was conducted to each of the previously constructed bridges, while three visits were conducted periodically over a period of one year following placement in an effort to determine rate of cracking over time. Tables 15 and 16 below display the dates for site visits to all bridges.

Table 15. Site visits performed on previously constructed bridges.

Bridge	Site Visit (2008)	Approx. Age of Rail in Months
Bethel	May 28	20
Chester	July 25	46
S. Burlington	September 2	24
Corinth	May 23	44
Lyndon	August 14	45
Morristown	July 1	72
Tunbridge	May 28	17
Underhill	September 2	82

Table 16. Site visits performed on newly constructed bridges.

Bridge	Site Visit	Approx. Age of Rail in Months
Barton	January 16, 2008	2
	May 3, 2008	6
	September, 2008	10
Randolph	November 7, 2007	1
	May 28, 2008	7
	September, 2008	11
Warren	November 7, 2007	1
	January 17, 2008	3
	June 17, 2008	8
	September, 2008	12

Due to potential safety hazards associated with conducting crack mapping evaluations, only the parts of the rail facing the roadway including the top and any in-window cracking was recorded. It is also important to note that several individuals were involved in recording data for this project. This may result in some variability within the data sets due to differing personal interpretations with respect to crack location and width. Therefore, the numeric results of this study are not to be taken as absolute values but rather as estimates to determine the trends present in the eleven structures evaluated.

4. RESULTS AND ANALYSIS

Overall, data collection revealed that all of the rails incorporated within this study exhibited some extent of cracking. Considering the relatively young age of the sample population, this result is disconcerting. This is easily demonstrated through the utilization of a simple crack density calculation. Table 17 below summarizes the total number of cracks present on each lane of each bridge rail, the structure length, and the number of cracks per foot of bridge. It may be important to note that the structure length may not exactly coincide with the length of the rail itself.

Table 17. Number of cracks per linear foot on all bridges. Numbers in bold represent bridges with windows.

Bridge	Lane	Total # of Cracks	Structure Length (ft)	Cracks per Foot
Bethel	US	53	183	0.29
	DS	72		0.39
Chester	US	140	112	**1.25**
	DS	137		**1.22**
S. Burlington	US	371	298	**1.24**
	DS	268		**0.90**
Corinth	US	55	72	0.76
	DS	39		0.54
Lyndon	US	110	172	0.64
	DS	83		0.48
Morristown	US	78	148	0.53
	DS	72		0.49
Tunbridge	US	212	184	**1.15**
	DS	206		**1.12**
Underhill	US	87	86	**1.01**
	DS	93		**1.08**
Barton	US	95	114	**0.83**
	DS	78		**0.68**
Randolph	US	132	204	**0.65**
	DS	168		**0.82**
Warren	US	0	64	0.00
	DS	27		0.42
	Total:	2576	3274	0.79
	Windows Total:	1987	1996	1.00
	Solid Total:	589	1278	0.46

The overall average crack density for all rails is 0.75 cracks per foot (or one crack every 1.3 feet), with an associated standard deviation of 0.34 cracks per foot. If the six bridges with windows and the five solid sections are averaged independently, the results are shown in the last two rows of Table 17. For rails with windows the number of cracks per foot more than doubles than for solid rails, 1.00 (or one crack every foot) to 0.46 (or one crack every 2.2 feet). While this shows the total number of cracks is far more drastic in the windowed rails, it is not quite that simple.

Due to the geometric nature of the windows, cracks form at the corners of the windows (or top of the arch if the window shape is arched). In general, cracks more readily form at the location of the smallest cross-sectional area, otherwise known as a weak point or area of increased stress concentration, such as at the top or bottom of a window. A large amount of windows were found to have cracks at both a top corner and a bottom corner. These cracks were counted as 2 individual cracks. If, instead of a window, this were a solid section with a crack extending the height of the rail from top to bottom it would only have been counted as one crack. Therefore, this methodology most likely accounts for a greater amount of cracking in windowed sections as compared to solid sections.

A correction factor was developed to normalize window rails to compare with solid rails. This was accomplished by counting a crack located at the top and bottom corner of a window as one crack. Table 18 shows the adjusted crack densities for the six bridges with windows.

Table 18. Total and adjusted crack densities for rails with windows.

Bridge	Lane	Total Cracking (cracks/foot)	Adjusted Cracking (cracks/foot)
Barton	US	0.83	0.76
	DS	0.68	0.58
Chester	US	1.25	0.79
	DS	1.22	0.79
Lime Kiln	US	1.24	0.94
	DS	0.9	0.59
Randolph	US	0.65	0.56
	DS	0.82	0.63
Tunbridge	US	1.15	0.97
	DS	1.12	0.93
Underhill	US	1.01	0.7
	DS	1.08	0.73
Average Crack Density:		**1.00**	**0.75**

Overall, this normalization technique reduced the average number of cracks per foot for bridge rails containing windows from 1.00 to 0.75 (or one crack every foot to one crack every 1.3 feet) which is still considerably greater than 0.46 cracks per foot for solid sectioned rails. Some bridges showed considerable amounts of change in crack density with this method, most notably Chester and the Lime Kiln bridges, both reducing the density of cracking by about a third. The two bridges affected the least by this methodology were Barton and Randolph, possibly due to the fact that those are the two newest bridges and have not had the chance to develop as many cracks both above and below associated windows as the other bridge rails.

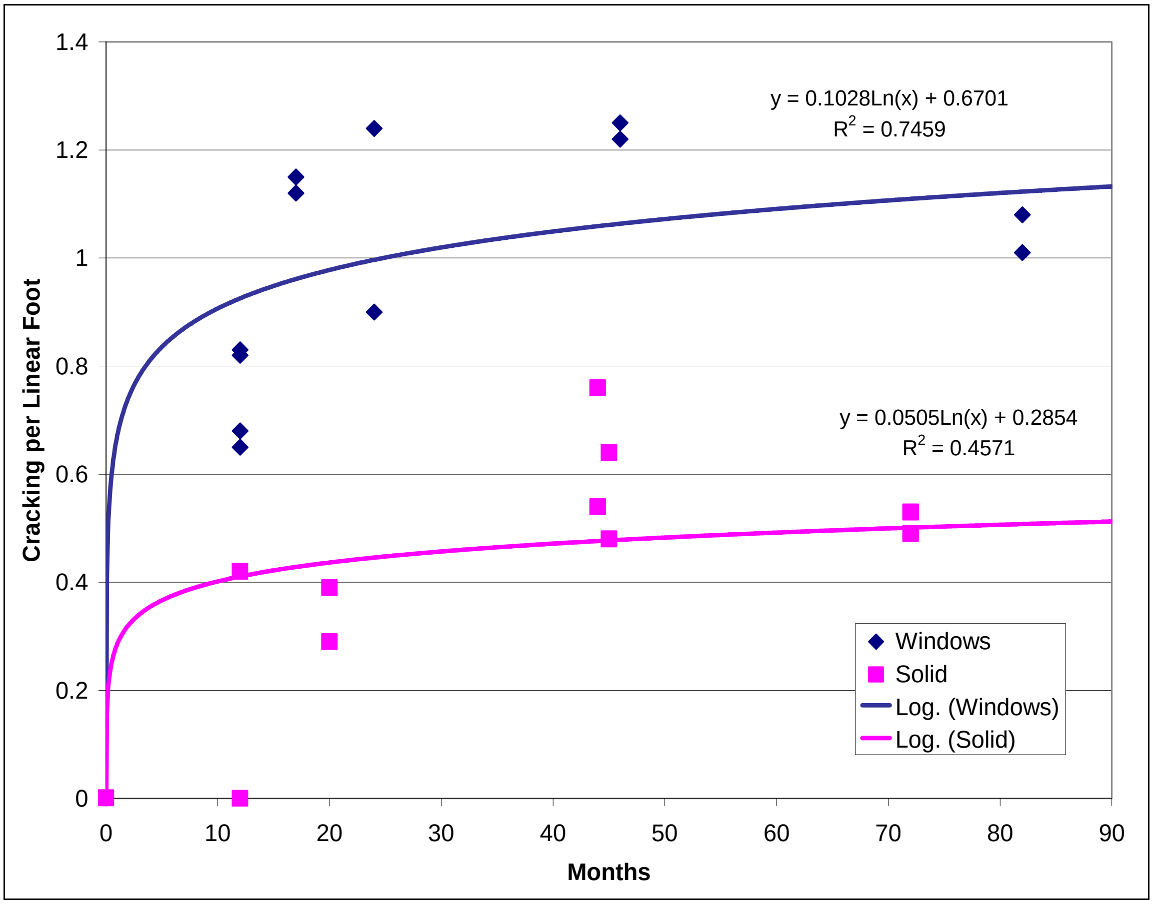

Figure 13. Cracking versus age plot for windowed and solid rails.

When the amount of cracking in each rail (downstream and upstream) for each bridge is plotted versus age, it is apparent there is a large difference in the amount of cracking between solid and windowed rails. In Figure 13, the pink line represents solid sections, which in almost every case have lower cracking values than the windowed (blue line) rails. Trend lines have been placed on the plot for each group, representing possible relationships between age and cracking. It is important to note that according to the best fit trend line, rails containing windows display a consistently greater amount of cracking over time at roughly 0.6 more cracks per foot (or one crack per 1.7').

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When the same plot is produced as Figure 13, with normalized crack densities for rails containing windows as opposed to the original density of cracking, the outcome is as shown in Figure 14 below. The two sample populations appear to display a similar amount of cracking as some overlap is observed. However, based upon the best fit trend line, rails containing windows still displays a greater amount of cracking at an average of 0.3 cracks/foot as compared to solid section bridge rails. This represents a 50% decrease as compared to 0.6 cracks per foot as shown in the previous figure. It is interesting to note that the rate of change of the new window trend line (the coefficient on the natural log term) has lowered and now more closely matches that of the solid rail trend line. Due to these two items, it can be hypothesized that the onset and rate of cracking for windowed versus solid rails will be approximately the same, except one can expect roughly 0.3 extra cracks per foot to form along rails with reduced cross-sectional areas due to the existence of the windows.

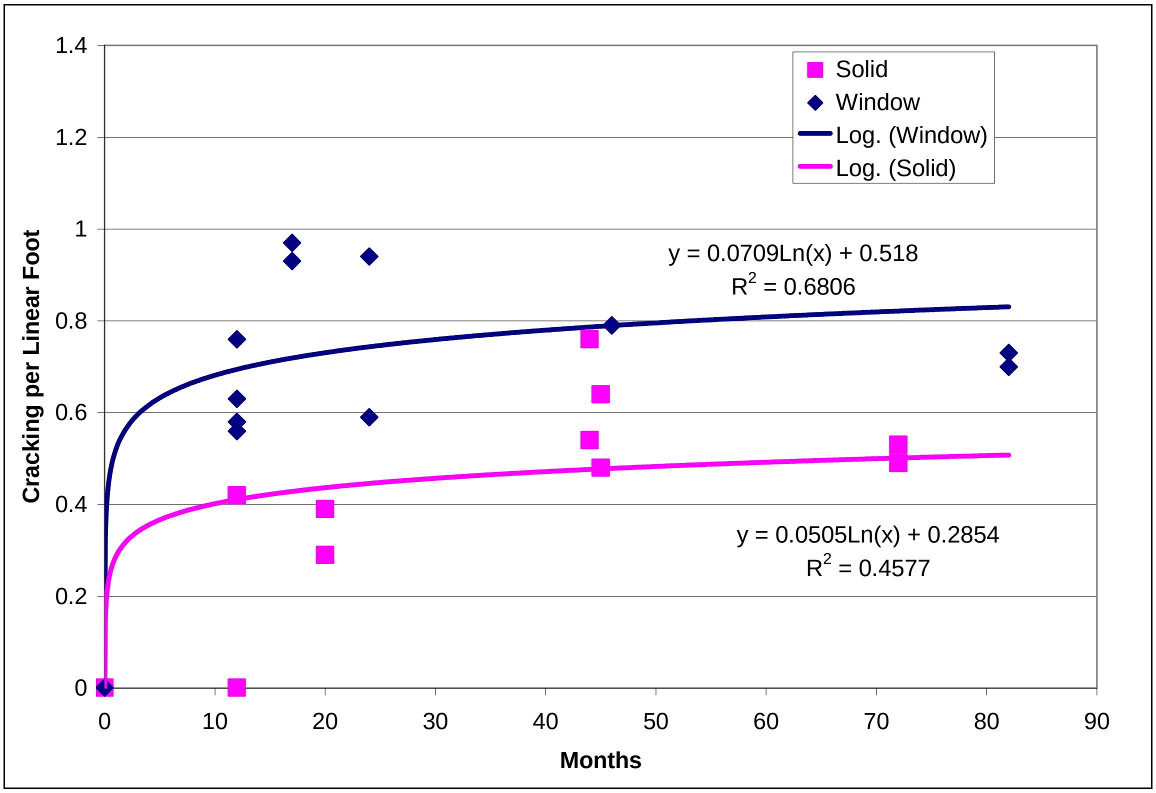

Figure 14. Cracking versus age plot for solid rails and adjusted values for windowed rails.

4.1.Recently Constructed Structures

For the three bridges cast following implementation of this study through their first year of service life, it was possible to examine the origin and development of cracking over time as shown in the plots below. These plots are provided and discussed in the following three sections.

4.1.1. Barton

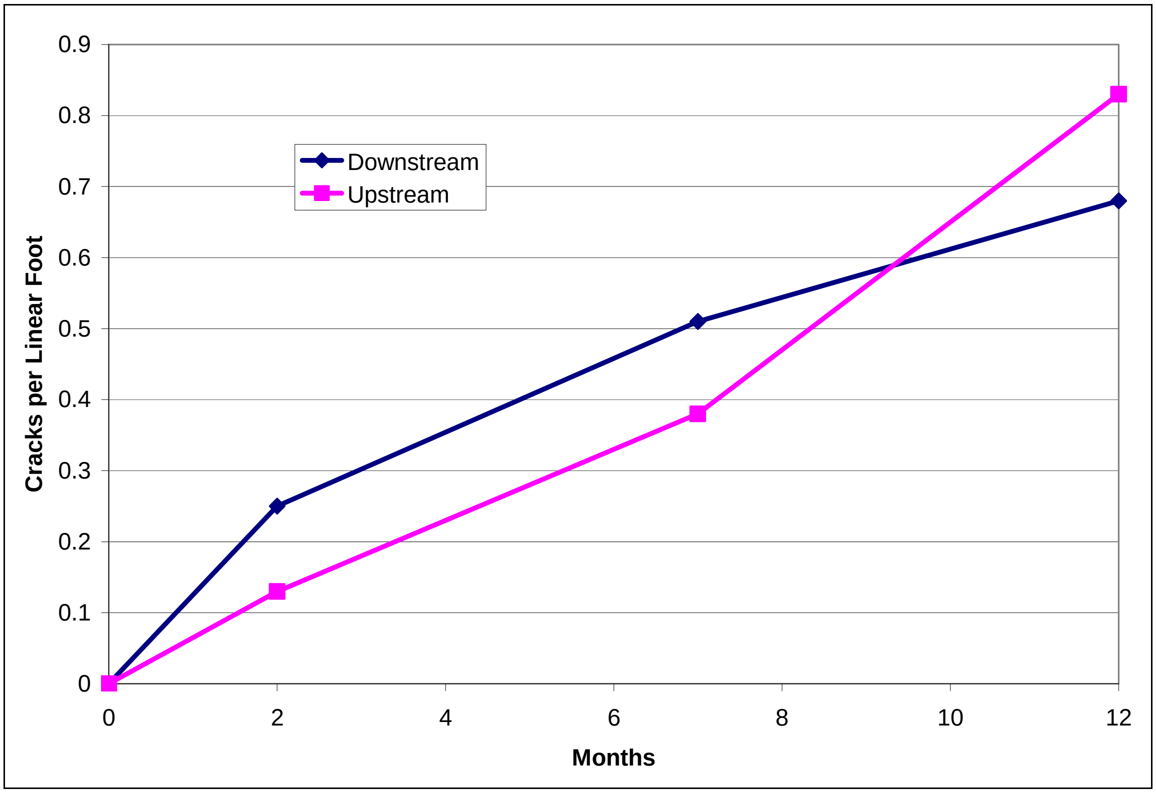

Figure 15. Plot of cracking over time for the Barton bridge rails.

The number of cracks along the Barton bridge rails, documented over the first year following construction, increased consistently between site visits as shown in Figure 15. On average, the amount of cracking along each rail has increased at a rate of about 0.06 cracks per foot per month, resulting in the average of 0.75 cracks per foot after 12 months of service. The crack widths, however, did not increase significantly, especially when compared to other bridges as will be discussed in the next section. During the five month period between the second and third site visit the widest cracks increased from about 10 mils to only 20 mils. The total count of the largest cracks was minimal compared to the total number of cracks overall; 10 out of 173 cracks were found to be greater than 20 mils, or 5.8%.

It is important to note that the concrete mix for this bridge contained a shrinkage control admixture, Eclipse Plus, as part of an experimental evaluation. It is possible that the admixture reduced the severity of the crack widths. However, the data collected provided evidence that the admixture did not eliminate or even reduce the amount of cracking along the rails, as it increased steadily over the year. If it were assumed that the admixture in fact performed well in limiting shrinkage cracking as anticipated, it would therefore be logical to believe that the cracking along the rails is then due to loading or other factors.

4.1.2. Randolph

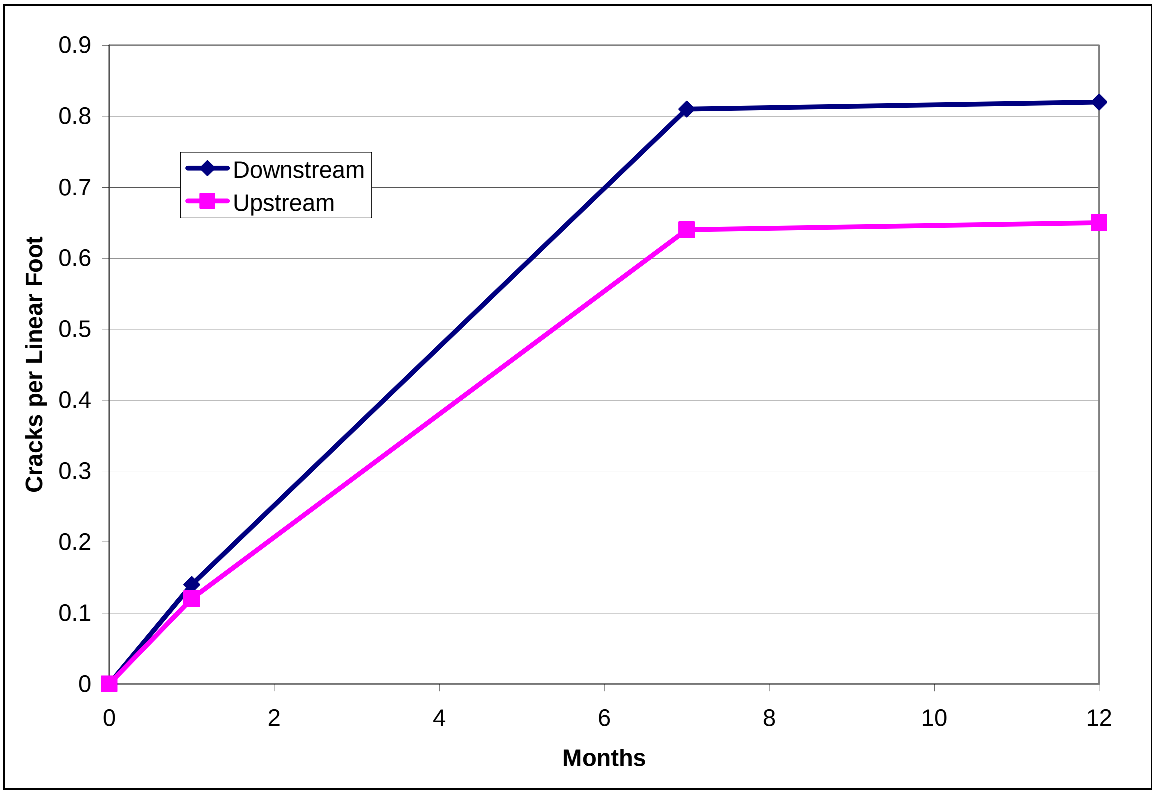

Figure 16. Plot of cracking over time for the Randolph bridge rails.

The Randolph bridge rails show a distinctly different trend than did the Barton rails, as displayed in Figure 16. The amount of cracking quickly increased to very similar levels as Barton, sometime within the first seven months to a similar average of 0.73 cracks per foot. In the subsequent five months, however, the total cracking barely increased, with only a total of only 2 more cracks reported along each rail. This would seem to indicate that maximum crack propagation occurred by the 7th month. However, it is important to note that there was a significant increase in crack width between the second and third site visit.

The widest cracks present at seven months following construction were on the order of 20 to 30 mils (as opposed to similar widths along the Barton structure one year following construction) and there were very few of these largest cracks (5.7% had a width of 20 mils plus). Once the rails reached one year of age the largest cracks were up to about 50 mils, with these being numerous and cracks in the aforementioned 20 to 30 mil range commonplace (13% of cracks were greater than 20 mils, while 2.7% of cracks were greater than 35 mils). It stands to reason that wider cracks allow for the penetration of contaminates and as well as water reducing the overall service life of the rails. It is also important to note that crack surveys were done during different seasons and varying weather conditions. Seasonal conditions may influenced the width of the cracks as the rails would likely contract during winter months and expand during summer months. A longer evaluation period, over several years, would be needed to determine if seasonal conditions influenced our findings. However, the most likely explanation for this phenomenon is that the cracks originated as small shrinkage, loading, or miscellaneous cracks. Then, over time, dynamic loading on the bridge in conjunction with the rigid concrete design, generated a cyclic loading pattern along the bridge rails, stressing and relaxing them, causing the cracks to increase in width over time.

Figure 17. Cracking in Randolph rail, sidewalk, and curb.

Evidence that many cracks may be due to loading, and not simple shrinkage, is shown in Figure 17. This photograph depicts a crack beginning (or ending) at the bottom right corner of the center window in the rail and migrating downward to the concrete and continuing across the width of the sidewalk and finally across the top face of the granite curb. It is most likely due to stresses generated by periodic loading, as a shrinkage crack would be material dependent and therefore would not propagate into other structures cast at differing times and different materials. Several of these cracks are present on both the upstream and downstream sidewalks, all propagating through the rail as well and some through the curb. Since there is an asphalt overlay present on the bridge, it is impossible to determine if the cracks continue into the bridge deck. Given the transverse locations of cracks on either bridge rail, it is plausible that some of these cracks may continue from one side of the bridge to the other. Out of all the bridges in the study, the Randolph structure has the highest AADT (16680) and truck traffic (1001), so it is subjected to large and repetitive loadings.

4.1.3. Warren

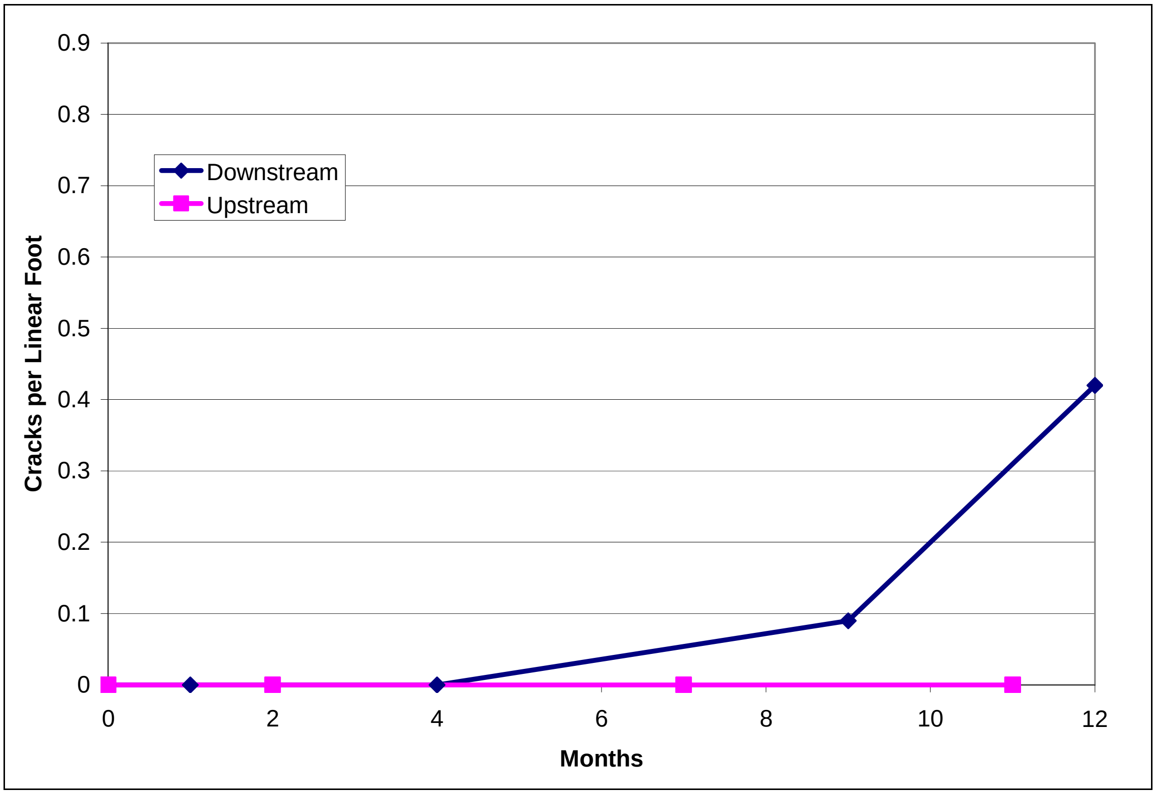

Figure 18. Plot of cracking over time for the Warren bridge rails.

The Warren bridge rails displayed yet another different trend from the other bridges, as can be shown in Figure 18. As of the eleven month visit, the upstream rail had yet to show any signs of surface cracking. As a note, the reason the upstream rail is eleven months old, while the downstream is twelve, is that the upstream rail was cast roughly six weeks later than the downstream rail. On the other hand, cracks did not begin to propagate along the downstream rail for at least the first four months following construction but since has started cracking at a higher rate, up to a total of just over 0.4 cracks per foot. This is approximately a 48% reduction in the number of cracks as compared to the bridge rails in Randolph and Barton. However as discussed previously this is a solid section and one would expect roughly half as many cracks to be present due to the geometric differences. The cracks that are now present on the downstream rail are all hairline cracks, very narrow when compared to the crack widths along many of the Randolph cracks.

The Warren bridge is the shortest bridge in the study at 64 feet and has the second lowest AADT at 1200 (Barton is the lowest at 740). Both of these factors may contribute to lower cracking due to loading (however, AADT, as discussed later, seems to have little affect on its own). Upon analysis it was discovered that the concrete mix design for this bridge also included the Eclipse Plus shrinkage control admixture as was specified in the Barton bridge. While this bridge was not part of the study incorporating this, it is possible that the admixture may have had a greater impact on this bridge than the Barton bridge due to its size or its design as a solid sectioned rail.

4.2.Corollary Analysis

4.2.1. Directional Analysis

Prior to the implementation of this study, it was hypothesized that the orientation of the rails may influence the occurrence of cracking. For example, rails with facing in the southerly and westerly direction will receive greater direct sunlight throughout the day as opposed to rails with a northerly or easterly direction. Therefore, they are subject to larger diurnal cycles (or hot and cold cycles) and associated stresses. This may have some impact during the curing process. In addition, during winter months, southern facing rails may be subjected to a greater number of freeze/thaw cycles potentially subjecting the rails to freeze/thaw damage due to the any infiltration of water into preexisting cracks or the formation of new cracks generated by excessive stresses.

Table 19. Orientation from north for all rails.

Bridge	Lane	Orientation from North
Bethel	US	137
	DS	43
Chester	US	82
	DS	98
S. Burlington	US	102
	DS	78
Corinth	US	63
	DS	117
Lyndon	US	135
	DS	45
Morristown	US	72
	DS	108
Tunbridge	US	40
	DS	140
Underhill	US	120
	DS	60
Barton	US	61
	DS	119
Randolph	US	85
	DS	95
Warren	US	34
	DS	146

Orientations of each bridge were documented referencing north as a baseline, as shown in Tables 3 through 14 (excluding 12) in Sections 2.1 and 2.2. Table 19 above summarizes the orientation of each rail in the study, with the degrees from north given as a normal to the roadway face of the rails.

The orientations listed above, when combined with their associated crack density values, results in the plot shown in Figure 19. A direction of 0 represents facing north and a value of 180 represents facing south; no distinction was made between east and west facing. Values vary widely throughout the 180 degrees. A parabolic trendline has been added to the data, showing a peak at approximately 90 degrees (representing east or west) and decreasing as the orientation goes to north or south. .

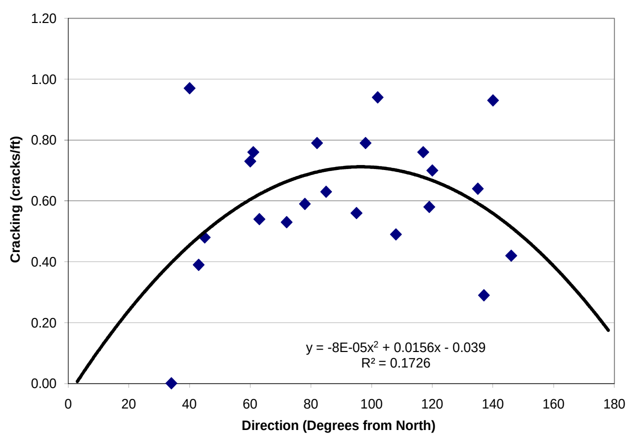

Figure 19. Plot of orientation versus crack density.

Other similar analyses were performed to examine the relationship between the onset and rate of cracking in comparison to bridge geometries and other functional aspects. For geometric relationships, length and number of spans were analyzed. This included an assessment of bridge length versus crack densities. A positive linear correlation between increasing lengths and an increased number of cracks was identified. The trend increases crack density on average by 0.08 cracks per foot per 100 feet of bridge length. This plot can be found in Appendix B, Figure B1. The number of spans in the bridges was also used to compare crack densities. Seven bridges in the study have one main span, three have two main spans, and only one has three spans. Based on this limited data, the trend shows about a 0.05 crack per foot density increase for each span added to a structure (Figure B2).

The two functional aspects of the bridges that were investigated include AADT (Figure B3) and daily truck traffic (Figure B4). Both of these explanatory variables displayed an inverse relationship meaning that an increase in daily and truck traffic produced a slight decrease in the density of cracking along the concrete bridge rails indicating they had very little effect on cracking. This finding is counterintuitive as an increase in traffic volume and truck traffic carrying a variety of vehicle weights would be expected to produce varying levels of stress along the structure and bridge rails assuming they display composite action.

4.2.2. Concrete Mix Designs

Each concrete bridge rail has a specific mix design containing various admixtures. Table 20 below summarizes the admixtures that have been incorporated into the concrete mix for the bridge rails for each project. Unfortunately the concrete mix design for the Morristown bridge was not available.

Table 20. Admixtures used in bridge rail concrete mixes.

Bridge	Adva 100	Adva 140	Daratard 17	Darex II	Micro-Air	Glenium 7500	Daracem 19	Daracem 65	Eclipse Plus	Flyash	Silica Fume
Bethel	●		●	●							
Chester	●		●	●							
Corinth	●		●	●							
Lime Kiln		●	●		●					●	
Lyndon	●			●							
Morristown											
Tunbridge	●		●	●							
Underhill					●		●	●		●	
Barton	●		●	●					●	●	●
Randolph	●		●	●							
Warren					●	●			●	●	●

All of the admixtures listed in Table 20 have unique properties and uses. Appendix C contains brief descriptions along with reported advantages. Information has been extracted directly from the product data sheets. From Table 20, it is apparent that the most common admixture combination that is used in rails is Adva 100, Daratard 17, and Darex II, as 5 of the 10 bridges with entries in the table use these constituents.

A regression analysis was performed to determine if any of the admixtures or the amount of cement in each rail affected the number of cracks that were observed. Table 21 displays the amount and type of cementitious material in each of the bridge rail's concrete mix designs. Due to the small sample population, especially with regards to certain admixtures of which were only incorporated into one of the mix designs, the results obtained cannot be statistically supported.

The results of the analysis suggest that the two main contributors to cracking amongst these parameters may be the Eclipse Plus shrinkage control admixture and the Daratard 17 initial set retarder. The Eclipse Plus, as expected, was shown to reduce the amount of cracking when used in the concrete by 0.2 cracks per foot (or one crack per 5'). The Daratard 17, on the other hand, actually increased the amount of cracking by 0.17 cracks per foot (or one crack per 5.9'). The amount of cement used, along with most of the other admixtures, had a neutral affect on the actual amount of cracking. Again, these results only represent this small study set and more analysis on a larger amount of projects would need to be performed in order to substantiate their validity.

Table 21. Total cementitious content of the bridge rail concretes.

Bridge	Type of Cement	Amount of Cement (lbs/yd^3)
Bethel	II/SF/Slag	705
Chester	II/SF	800
Corinth	II	660
Lime Kiln	II/SF/FA	705
Lyndon	II/SF	705
Morristown	n/a	n/a
Tunbridge	II/SF/Slag	705
Underhill	II/SF/FA	710
Barton	II/SF	611
Randolph	II	660
Warren	II/SF/FA	705

4.2.3. Cost Comparison

Based on discussions with both Agency's Construction Section and contractor personnel, it is apparent that there is a considerable increase in cost, labor, and time constructing bridge rails as opposed to alterative bridge rails.

A cost comparison was performed utilizing previous contract bid results furnished by the Structures section. On average, the lowest serviceable level of steel rail (SBR6) costs around $50 a linear foot, however this is not a rail of choice for designers in the Structures Section. Two types of steel rail are regarded as comparable to concrete bridge rails (which are rated as NCHRP test level 2), the SBR7 and NETC rail. The cost for these two rails are around $100 and $125 per linear foot, respectively, for materials and labor. Texas rail, on the other hand, costs on the order of $250 per linear foot to produce, generating, at a minimum, a 100% cost increase over steel bridge rail. In addition to the straight cost increase is the fact that it takes several times longer to cast and cure, thus creating a delay in the construction sequence on other adjacent areas of the projects.

5. SURVEY OF STATES

A survey was compiled and distributed to other regional states in order to gauge their concerns and experience with aesthetic concrete bridge rail cracking as well as to document their methods of the placement of such structures. The survey was submitted to regional research managers at State DOTs to answer or pass on to the appropriate personnel for their response. The survey contained the following ten questions:

1. Has cracking in concrete bridge rails been raised as an issue or problem? If so, to what extent and what type of cracking?

2. If yes, has a similar study been done to the one we are undertaking and what were the recommendations?

3. What type of concrete bridge rails are currently being utilized, e.g. Texas aesthetic, etc.?

4. What type of concrete mix designs are being used in these structures?

5. What construction procedures are being used for casting these rails, e.g. cast in place, slipforming, etc.?

6. What curing methods are being used or specified for these and similar vertical structures?

7. Does your agency specify epoxy coated reinforcement for the rails?

8. Are coatings/sealants being specified for the rails?

9. What surface finish method is used for the rails? E.g. rubbed, sacked, other.

10. Are deicers applied to the bridge decks and if so which are predominantly used? E.g. salts, calcium magnesium acetate, others (urea, heavy salt brine, etc).

Given the small survey distribution, the number of surveys returned was quite high, with seven submittals. The respondents were from: Delaware, Florida, Maryland, New Jersey, New York, Quebec, and Rhode Island.

Table 20 below summarizes all answers from six of the seven respondents. Only Florida is not included, as their response detailed the fact that they have not recently used any type of concrete bridge rail.

Table 22. Summary of survey responses from other DOTs.

	Delaware	Maryland	New Jersey	New York	Quebec	Rhode Island
1	Yes, shrinkage cracks	No, we use very few of these rails	Not observed as a significant problem	Yes, pronounced with Texas rails	Yes, but not on aesthetic, only on straight rails	No, not to any significant degree
2	No study, but cracks are sealed with Sika products	N/A	N/A	No	Yes, City of Montreal requires construction joints 2m apart, Ministry does not	N/A
3	Straight walls, F-Shaped, Texas	Concrete parapet walls with aluminum rail on top	NJ barrier shape and 32" high concrete straight section	Texas, Jersey, Single slope, F shape, vertical parapet	Mainly New Jersey type	None, limited repairs of historic bridges, no standard detail
4	4,500 psi criteria	4,500 psi for cast-in-place	High performance concrete mix design	HPC and A	Type XIII concrete, 50 MPa, with ternary cement (including silica fume)	HPC
5	All, with slip forming at contractors request	Both cast-in-place and slip formed	Both cast-in-place and slip formed	Cast-in-place, slip cast, or precast	Cast-in-place, concrete cover=75mm, fabric inside cofferdams	Mostly cast-in-place, some slip forming and precast
6	Normal water methods and/or compound	Wet burlap or clear curing compound	Spray applied curing compounds	Curing compound or leaving forms in place	Cofferdams kept in place for 7 days, 3 if temp above 15° C	Wet curing, curing compounds, or both
7	Yes	No	Epoxy coated used primarily, galvanized and stainless steel clad also permitted	Yes	No, galvanized rebar is specified	Yes
8	Yes	N/A	Coatings and sealants are not normal practice but are permitted	Varied by region and designer preference	No	Water-based epoxy film former
9	None	N/A	Rubbed finish	Only holes greater than 15 mm require remediation. No rubbing except to remove form lines	Light grade is required with high pressure water or wet abrasive	Form finish
10	Salt and Sand	Salts	Primarily calcium magnesium	Salt, salt brine, calcium chloride, magnesium chloride, potassium acetate	Sodium chloride	Sodium chloride and sand mix

As a whole the responses are not unexpected. Most states have very little experience using aesthetic Texas rails. Of the states that have been using them in a limited fashion, there seems to be at least a known problem of cracking in concrete bridge rails.

No definitive relationship between rail cracking and other variables (curing, finishing, and salting methods, etc.) could be found with this limited response. One item of importance that both the Ministere des Transports Quebec and the New York Department of Transportation mentioned is the fact that through their studies they found that the use of contraction joints every few meters to 20 feet may provide some benefit. Contraction joints are joints that do not contain reinforcing, thus each rail section can expand and contact independently thereby isolating stresses.

6. CONCLUSIONS AND RECOMMENDATIONS

Through the observation and study of eleven bridge structures containing concrete bridge rails throughout the State of Vermont there appears to be a significant occurrence and continued increase of cracking most likely attributed to one or several of the following factors including design, concrete mix, casting and curing methods as well as dynamic loading. All eleven bridge structures incorporated into the sample population can be considered fairly new as the oldest was built in 2001. Three bridges within the sample population were constructed during this study allowing for periodic site visits to document construction practices, gain insight from Agency personnel and contractors, as well as document the onset and rate of cracking over a one year period. Overall, cracking was observed along all concrete bridge rails. Such cracking in concrete rails, and concrete in general, allow for the ingress of contaminants, such as chlorides, into the structure and facilitate corrosion of the reinforcing steel and ultimately deterioration of the concrete. Therefore as more cracking is present in a rail, the lower the longevity of the rail.

Rails with reduced cross sectional areas, otherwise known as weak points, were found to display a greater number of cracks along the rails as compared to solid sectioned rails. Crack surveys indicate that the density of cracking along rails containing windows (areas with reduced cross sectional areas) and solid sections rails were found to be 0.75 and 0.46 cracks per linear foot, respectively. Please note that the crack density of 0.75 for rails containing windows is a normalized value as described in Section 4. It is clear that solid sections do, in fact, appear to crack far less than other designs, however 0.46 cracks per foot is still a considerable amount; this is equivalent to one crack for every two feet of rail. Rails with areas of reduced cross sectional area are more susceptible to cracking, about 60% more according to data in this study. It is unknown what structural impact this may have on the stability of the rail over time. It should be noted that while rails containing windows are more susceptible to cracking, they are more visually appealing which may facilitate continued public support.

Analysis of crack densities versus the age of the rails indicates that cracking most likely occurs fairly rapidly upon concrete casting. Data pertaining to the previously cast rails reveals that the greatest increase in crack densities occurs during the first two years of service and then decreases and remains somewhat constant. What this data does not speak to is the possibility that although the number of cracks may not increase two years following construction, the width of the cracks may increase over time, well beyond the aforementioned period. The three structures that were monitored during and following construction provide some proof of this. The Randolph and Barton bridges both displayed rapidly increasing crack densities over the first year. The cracks on both bridges, but especially Randolph, increased in size over time even if there was not a marked increase in the number of cracks. With the limited duration of this study it is not possible to determine the length of time cracks continue to grow in width.

Miscellaneous corollary statistics unfortunately did not reveal any definitive reasons for decreases or increases in crack density along particular structures. Directional analysis, comparing the degrees from north from which rails faced, showed a small correlation. Greater orientation to a northerly direction displayed a lesser amount of cracking as compared to those oriented in the southerly direction. A possible reason for this is that rails facing a southerly direction are subject to more freeze/thaw events during winter months, due to sun exposure. Therefore a greater extent of expansion/constriction occur generating greater stresses thereby mitigating crack propagation. Other analyses, such as traffic and the number of spans a bridge is comprised of showed no measurable correlation. A larger sample population is needed to determine identify and validate any statistical trends.

A regression analysis of the significance of each concrete admixture and cementitious content of each rail determined that the addition of the Eclipse Plus shrinkage control admixture may reduce the amount of cracking in the rails by about 0.2 cracks per foot, while Daratard 17 may actually increase cracking by 0.2 cracks per foot. Most other admixtures, as well as the amount of cement present, were determined to have a neutral affect. Unfortunately this analysis cannot be considered statistically relevant due to the small sample size.

Limited information was extracted from the responses of various states and provinces that responded to a survey distributed as part of this study. In general most states have not noticed a problem due to limited use of aesthetic concrete bridge rails. A few states indicated that they have noticed a problem but have done little to try and quantify or alleviate the issue.

It is recommended that a restriction be placed on the general use of concrete bridge rails and be used only when necessary. The main reason that they are utilized is to satisfy requirements by local officials who request that reconstructed bridges retain their historic appearance. It should be attempted to keep these occurrences to a minimum, using this type of structure only when there are no other alternatives. All other uses of the rails should be postponed. It is also important that sealing the cracks is undertaken on a periodic basis early in the structure's life to exclude rapid migration of contaminants into the cracked section.

The limits on the use of aesthetic concrete rail should extend in time until further research can be devised, performed, and completed. One example of needed research in the area is concrete mix design in order to optimize the characteristics of the concrete to limit (or hopefully eliminate) the cracking phenomenon. A second example could be an examination of reinforcement practices and loading effects on high strength-low age concrete. Other research efforts could include the implementation and associated performance valuation of preformed concrete rail elements. These are rails produced in a shop setting under a controlled environment, and are held in this environment until completely cured or sealed. They are then installed in the field. It may be a good idea to attempt the placement of one of these on a bridge in the near future to determine if it can mitigate our cracking problems in a cost effective manner. Other possibilities for future research could include: a correlation between design deflections in bridges and the observed crack densities in rails and decks; the effect of increasing or decreasing the amount of reinforcing steel to the cross section of rail elements; and placement of rails so as to isolate their movements, independent of the deck, in order to alleviate cracking while still maintaining the crashworthy integrity of the rails.

Continuing to place materials on projects when there is a near 100% chance of cracking and possible failures should be weighed carefully as it leads only to dramatic increases in repair and maintenance costs in the future, as well as dissatisfaction for the users of the structures, who cannot help but display a decrease in support upon seeing flaws in newly constructed structures.

APPENDIX A

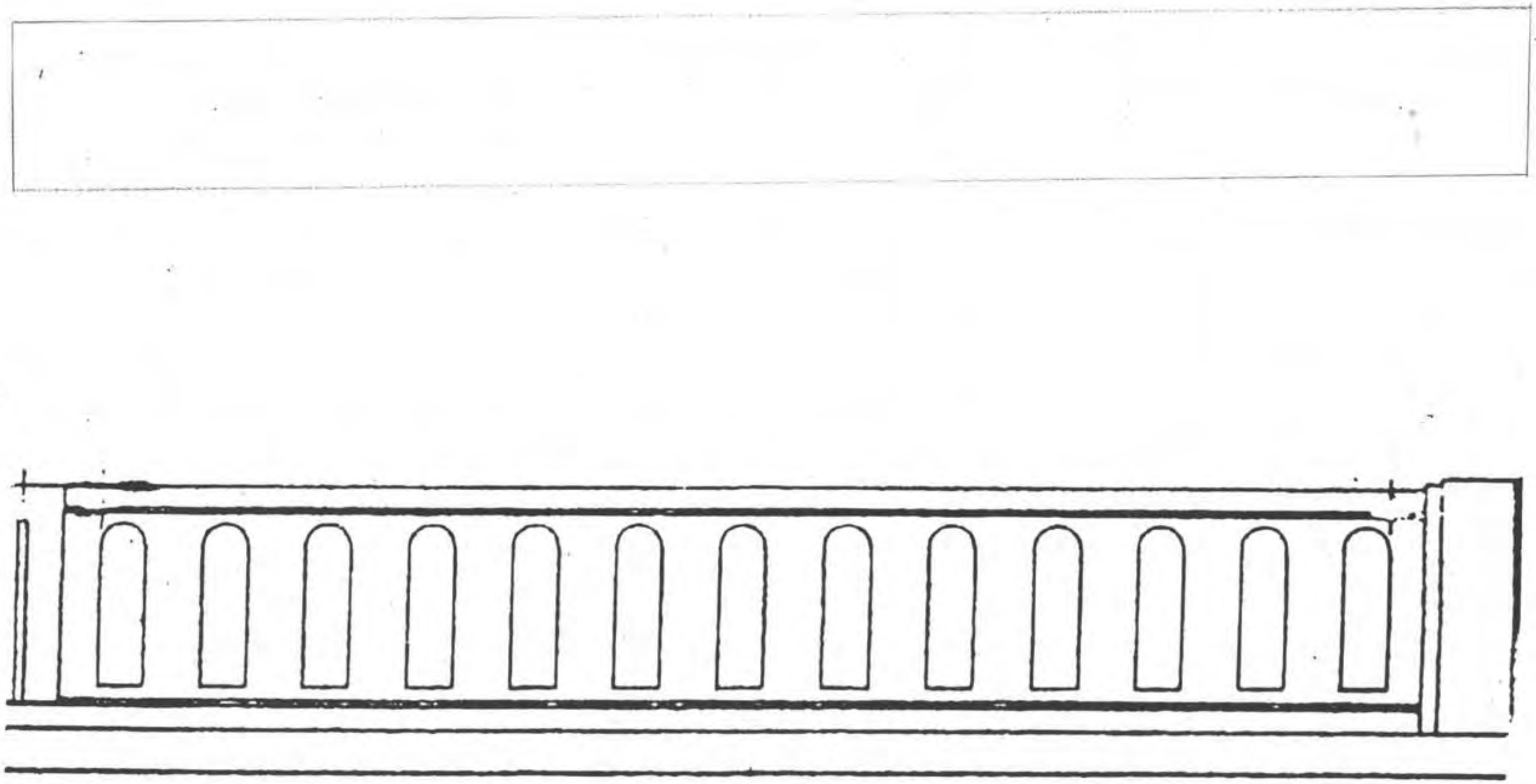

Figure A1. Blank crack mapping diagram of a Texas bridge rail.

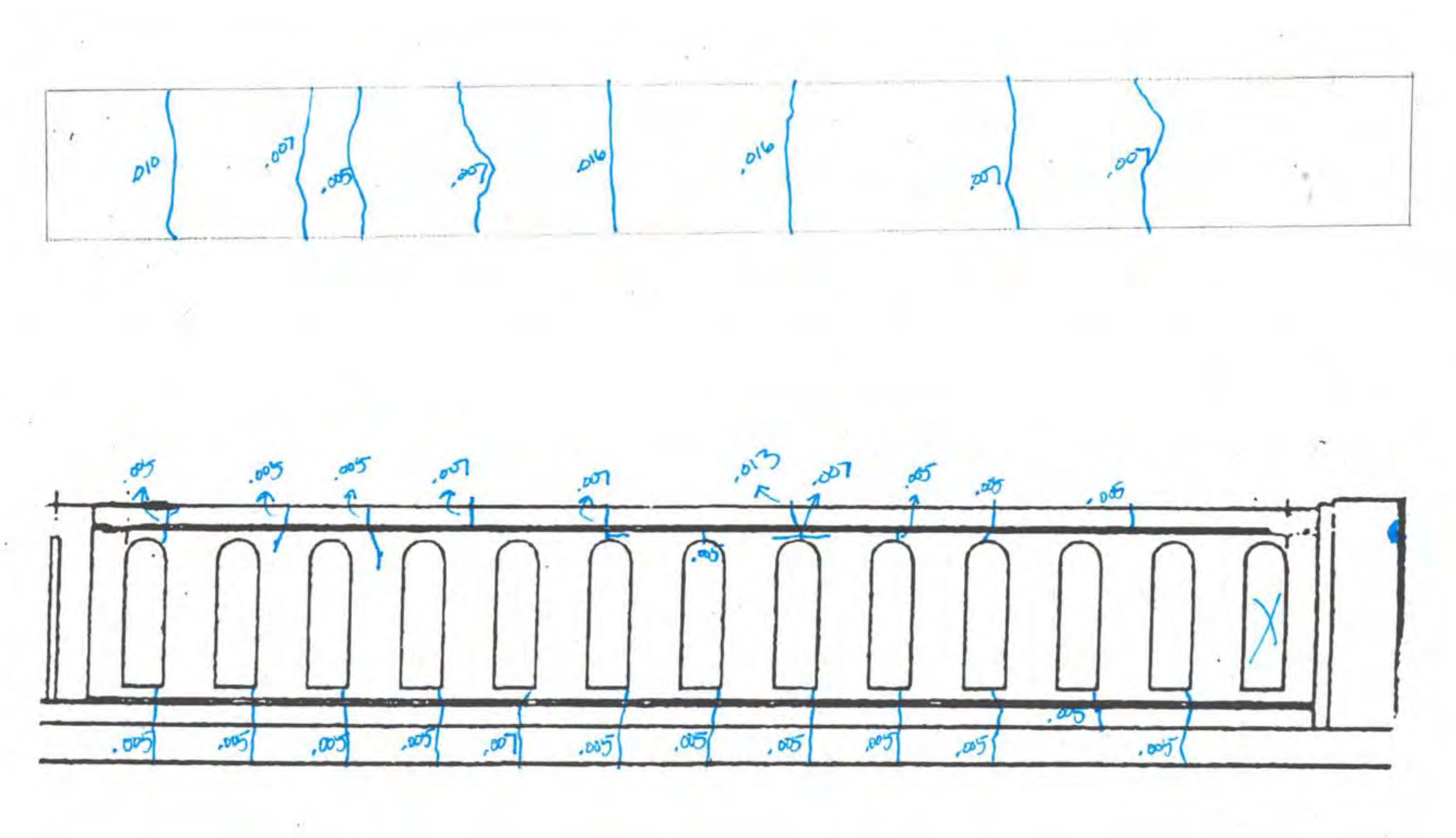

Figure A2. Completed crack mapping diagram of a Texas bridge rail.

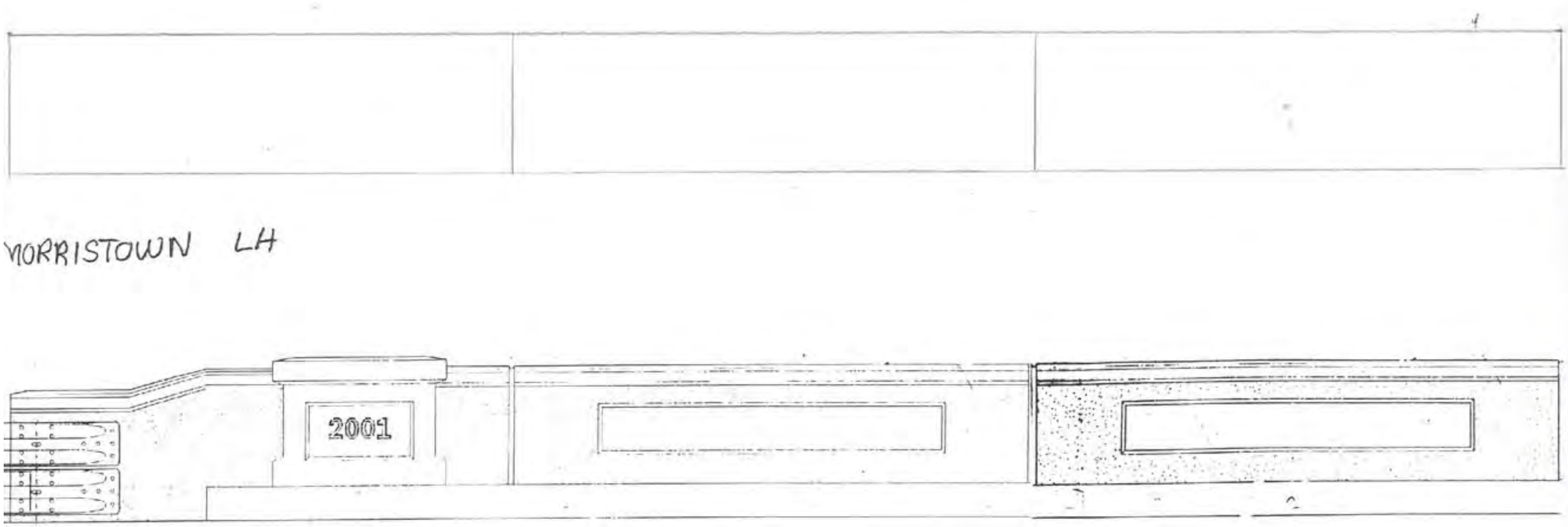

Figure A3. Blank crack mapping diagram of a solid bridge rail.

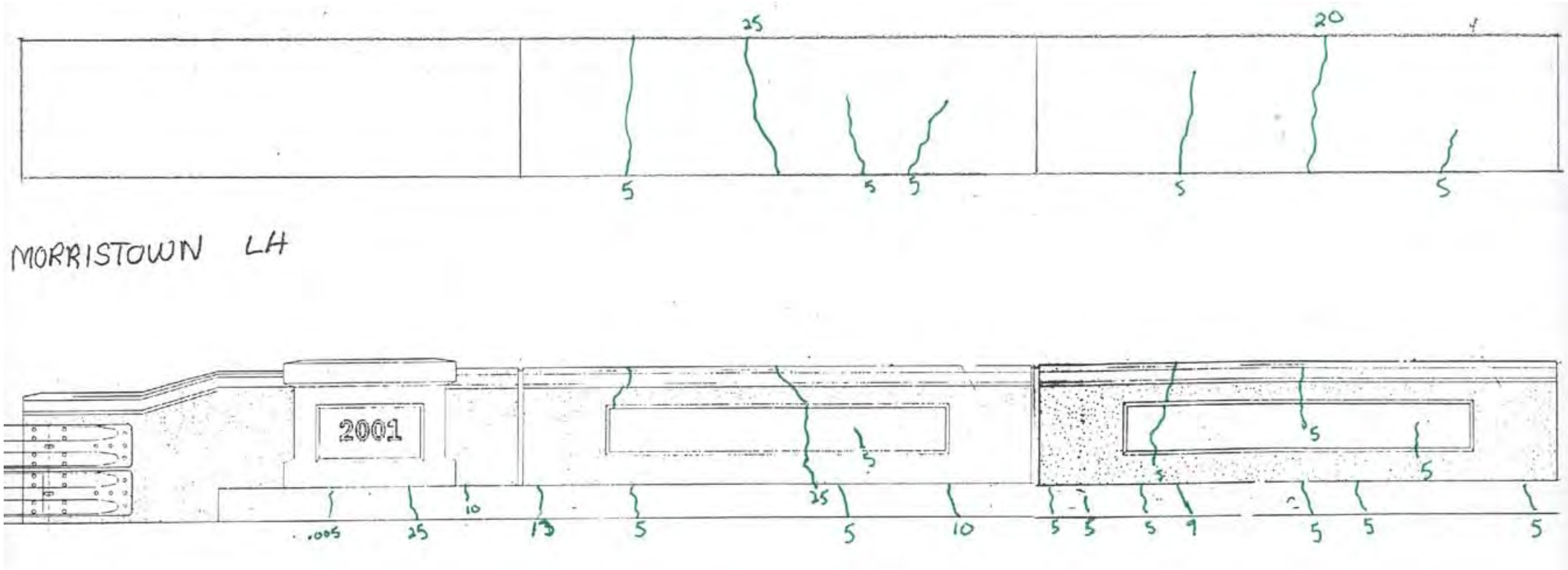

Figure A4. Completed crack mapping diagram of a solid bridge rail.

APPENDIX B

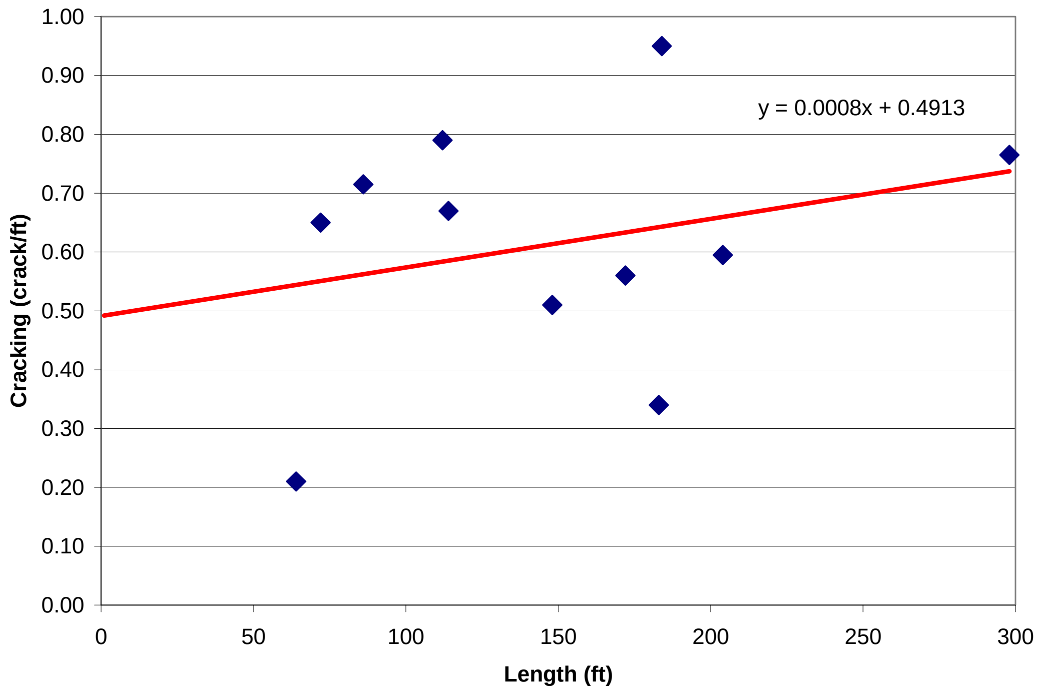

Figure B1. Plot of overall bridge length versus crack density.

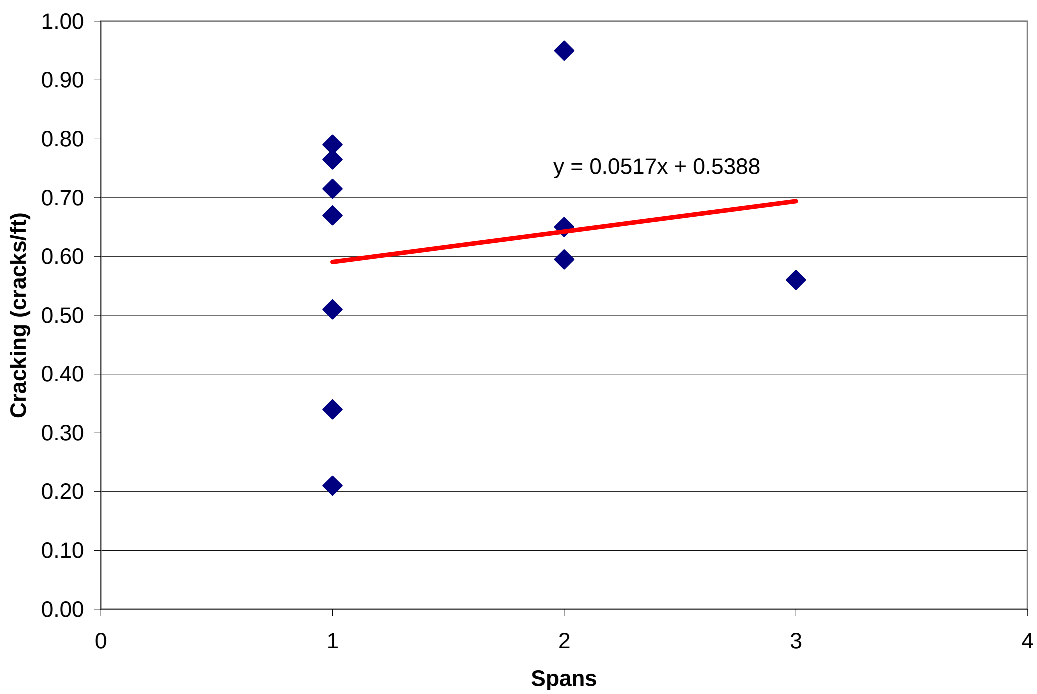

Figure B2. Plot of number of main spans versus crack density.

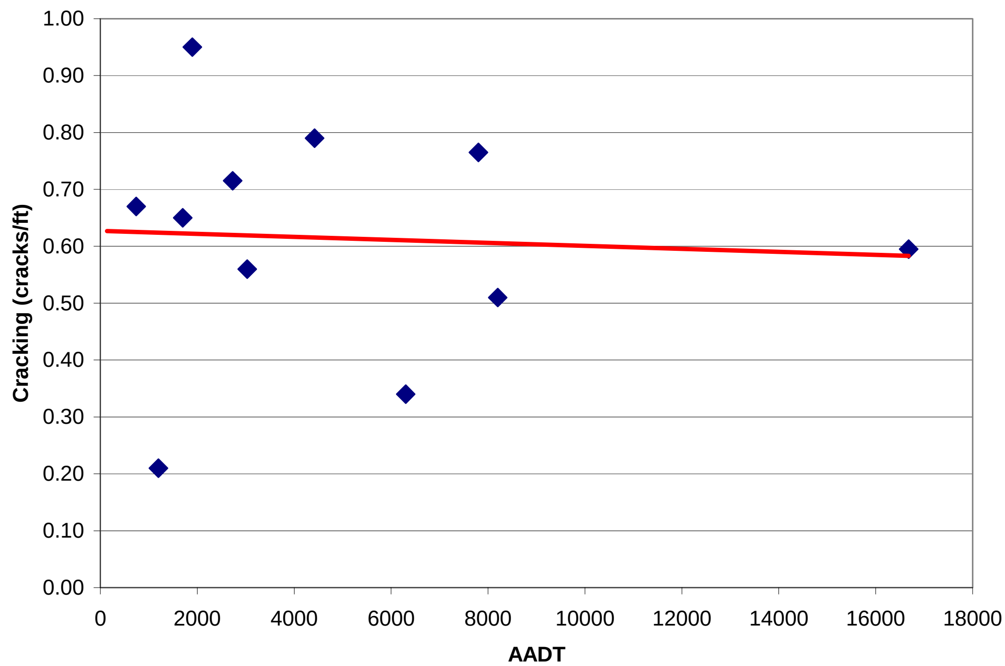

Figure B3. Plot of AADT versus crack density.

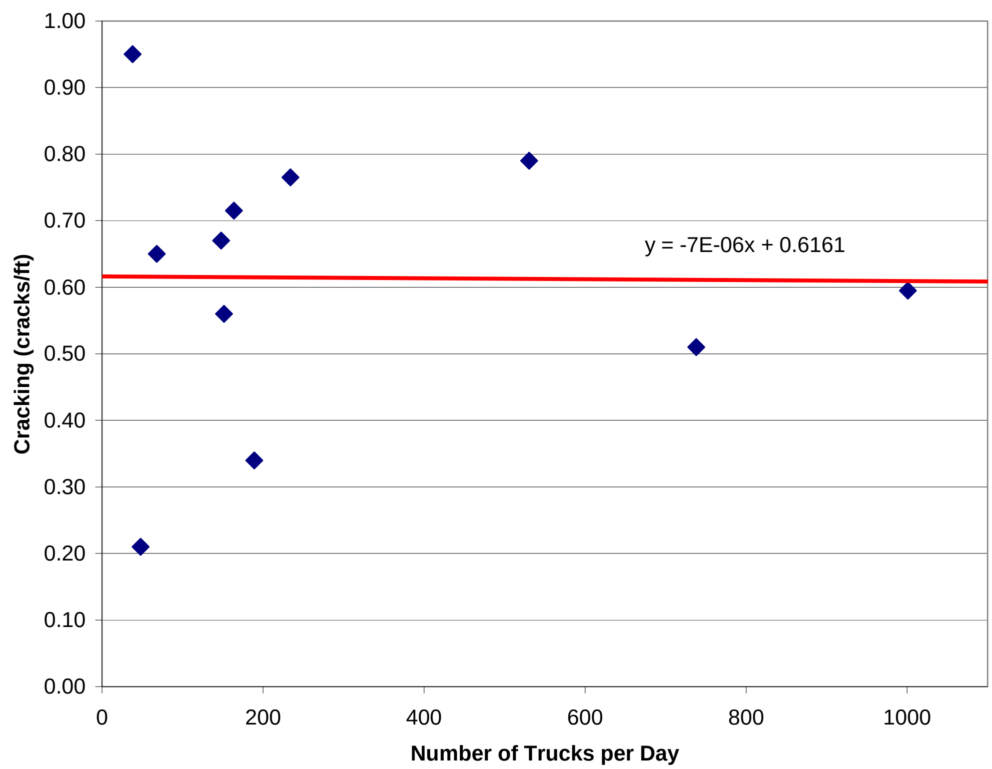

Figure B4. Plot of truck traffic versus crack density.

APPENDIX C

The following are brief descriptions of all admixtures that have been added to the concrete mixes for the bridge rails in this study. Descriptions and reported benefits have been extracted directly from the various companies' product data sheets. The manufacturer of each product is listed in parentheses.

Adva 100: High range water-reducing superplasticizing admixture. Contains no added chloride. (Grace)

- High slump concrete at very low dosage with no loss in strength
- Long slump life with near neutral set time
- Reduces water/cement ratio and still achieves the same degree of workability

Adva 140: High range water-reducing admixture based on polycarboxylate technology. (Grace)

- Consistent air entrainment
- Provides a superior combination of long slump life with near neutral set time
- Can be used as a high or mid-range water reducer

Daratard 17: Ready-to-use initial set retarder, aqueous solution of hydroxylated organic compounds. Weighs ~ 10.2 lbs/gal. (Grace)

- Extends the initial setting time of concrete by 2 to 3 hours at 70°
- Provides water reduction (typically 8 to 10%), which produces greater plasticity and workability

Darex II: Air-entraining admixture which generates a highly stable air void system for increased protection against damage from freezing and thawing, sever weathering, or deicer chemicals. It is a complex mixture of organic acid salts in an aqueous solution. (Grace)

- Improves air entrainment stability
- Effective in maintaining air content during longer haul times
- Aids workability to the mix and permits a reduction of water with no loss of slump
- Placeability is improved
- Bleeding, segregation, and green shrinkage are minimized

Micro-Air: Ready-to-use liquid air entraining agent for use in all types of concrete. (BASF)

- Improved plasticity (workability)
- Improved cohesiveness; reduces segregation and bleeding
- Improved ability to entrain and retain air in concrete; increases durability to damage from freezing and thawing

Glenium 7500: High-range water-reducing admixture, based on the next generation of polycarboxylate technology. (BASF)

- Excellent early strength development
- Controls setting characteristics

- Optimizes slump retention/setting relationship
- Consistent air entrainment
- Dosage flexibility

Daracem 19: High-range water-reducing admixture that is an aqueous solution of modified naphthalene sulfonate, which disperses the cement agglomerates normally found in a cement-water suspension. (Grace)

- High slump flowable concrete with no loss in strength
- Low water/cement ratio concrete
- At high slump, exhibits no significant segregation

Daracem 65: Mid-range water-reducing admixture specifically formulated to produce concretes with dramatically enhanced finishing characteristics and normal setting times. (Grace)

- Ultimate workability and finishability
- Neutral setting times
- Superior strength performance

Eclipse Plus: Liquid admixture for concrete which dramatically reduces the materials shrinkage due to drying. It has been shown that it reduces shrinkage by a much as 80% at 28 days, and up to 50% at one year or beyond. This level of shrinkage reduction, in well proportioned concrete mixtures utilizing quality materials has demonstrated to eliminate cracking due to drying shrinkage in fully restrained concrete. (Grace)

- Reduces surface tension
- Saves time and costly repairs
- Enhanced durability with longer usable life

Flyash: Product of the combustion of coal in large power plants.

- More durable finished concrete
- Produces high strength concrete that accommodates the design of thinner sections
- Permits design flexibility accommodating curves, arches, and other pleasing architectural effects
- Later-age strength gain
- Contributes to the aesthetic appearance of the concrete