# A Comprehensive Evaluation of High Friction Overlay Systems on Bridge Decks in Cold Climate Regions

A Thesis

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## Abstract

In recent history the Minnesota Department of Transportation has looked to improve the safety of bridges. One improvement method is placing high friction overlays (HFO) on bridge decks. The primary goal of this research was to complete a comprehensive study of different proprietary HFO systems placed on bridges throughout the state of Minnesota. Four different proprietary HFO systems placed on fourteen different bridges were studied.

Research was split into three separate tasks. The first task was laboratory testing on different aggregate sources used in HFO systems. Second, a two year field study was conducted including observation and testing of different HFO systems placed on bridges throughout the state of Minnesota. Testing was conducted to obtain the mean texture depth, bond strength, permeability, and friction values of the HFO systems. The third task was the comprehensive analysis of ten years of crash data encompassing before and after HFO systems were installed on the bridge decks. Crash characteristics analyzed included the accident time, weather conditions, bridge surface conditions, Average Daily Traffic, and severity of accidents.

Field testing and observations revealed that the extensive use of snowplows during winter months extensively abrade the high friction systems. This abrasion causes a reduction in the surfaces friction values and reduces the life of such a system. The analysis of crash data suggests that although there is a reducing trend in overall accidents, a reduction in accidents cannot be completely attributed to the use of high friction overlays. Furthermore, the presence of ice packed snow on HFO systems nullifies their high friction values. The reduction in crashes directly correlated to the installation of HFO systems is minimal. It is recommended that use of HFO systems in Minnesota be reevaluated.

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# Chapter 1: Introduction of the Project

## Introduction to High Friction Overlay Systems

Improving the safety of roadways has been an objective since the arrival of automobiles at the turn of the 20th century and the advent of paving roadways. Beginning in the 1990's the Minnesota Departments of Public Safety and Transportation began looking at ways to reduce serious injuries and fatal crashes on Minnesota roadways. In 2003 a program called Towards Zero Deaths (TZD) was initiated. It is an interdisciplinary safety program through the Departments of Public Safety, Transportation, and Health. With this program the state strives for zero deaths on roadways through education, engineering, enforcement, and emergency medical and trauma services. Over the past decade the TZD program has been a spearhead in traffic safety (MnDOT, 2014).With the harsh winter conditions that exist in the state of Minnesota there is a great need to provide safe driving conditions throughout the entire year. Over the last ten years the Minnesota Department of Transportation (MnDOT) has employed high friction overlays (HFO) as a possible anti-icing and anti-skid solution.

A high friction overlay is a thin overlay (less than one inch [2.5 cm]) which consists of fine aggregate embedded into an epoxy or asphalt overlay creating a pavement wear course. Epoxy overlays are generally constructed in two lifts in order to ensure that the aggregate does not settle to the bottom of the binder (Cargill, 2014). Asphalt overlays are constructed in a single lift (Midland, 2014). Although more expensive compared to a traditional overlay (asphalt or concrete), an HFO has several benefits: (1) lighter dead load, (2) fast cure time, (3) shallow depths eliminating the need to raise approach panels, (4) a waterproof wear surface, and (5) high skid resistance (Harper, 2007). Several different HFO systems have been placed on bridge decks across the state of Minnesota in order to observe and assess how high friction overlay systems work in cold climates and if they provide benefits for Minnesota's roadways.

## Need for this Study

With the continual strive for safer roads, MnDOT has invested resources into the use of HFO systems. In 2010 a report was published by MnDOT summarizing research conducted on SafeLane high friction overlays placed within the state. This past research studied three bridge locations for three years after the installation of SafeLane HFO systems. A more in depth summary of this study is found later in the literature review (Chapter 2), however one of the major conclusions in the report was the recommendation for further research of HFO systems (Evans, 2010). Funding for the current research was allocated based on this recommendation. Several additional HFO systems were placed in Minnesota between 2008 and 2011. This current research improves upon the past research by encompassing these additional bridges and conducting research several years after their installation. These conditions allow for a study encompassing more bridges, more testing, and longer time intervals since the installation of the HFO systems.

## Objective of this Study

The purpose of this project is to provide a comparative analysis of the different HFO systems with two objectives. The first is to investigate the HFO systems and provide a comparative analysis of the different systems that evaluates the safety aspects they provide. The second objective is to serve as a model for future research involving HFO systems and their benefits in the reduction of crashes.

In order to ascertain the benefits of placing an HFO system on a bridge deck, extensive research of each system and the multiple variables at each location is required. A comparative analysis can be provided through the observation, testing, and data analysis of different conditions and performances for each HFO system.

This study will look to answer several questions regarding HFO systems including:

(1) How do HFO systems perform in cold weather climates that exist in Minnesota?
(2) Do HFO systems provide benefits that reduce crash rates on bridges?

(3) Is there a correlation between surface friction values of an HFO system and their crash rates?

(4) Do the benefits that HFO systems provide to Minnesota bridge decks justify their use in the state?

## Organization of Thesis

This thesis begins by providing background information of the high friction overlay systems that are covered in this study as well as the locations across the state where the different systems were installed. An extensive literature review was performed to gain insight on previous research conducted on HFO systems in Minnesota as well as in other states (Chapter 2). Following the literature review, research was broken into three different tasks. Chapter 3 is a compilation of laboratory tests performed on different aggregate sources used in HFO systems. Chapter 4 is a two year field investigation which encompassed site visits to the different bridges with HFO systems. Field tests and extensive observations were conducted. Chapters 3 and 4 are broken down into two main parts. First is the scope of testing and observations completed; second the results of testing and observations are presented. Chapter 5 is a comparative analysis of crash data for the bridges with HFO systems. This analysis covers over a decade of crash data provided by MnDOT. The thesis culminates with a summary that ties together the results of the three tasks in order to make appropriate conclusions and recommendations for the entire study (Chapter 6). Following Chapter 6 are the References and Appendices. These sections provide sources, data, and calculations used to complete the research.

## Background Information

The following is a short overview of the four different proprietary HFO systems studied as well as a look at the different locations in Minnesota where the HFO systems were installed.

### High Friction Overlay Systems

Four types of proprietary high friction overlays are included in this study.

- **Poly-Carb HFO systems** (product of POLY-CARB, Inc., a subsidiary of The Dow Chemical Company) are epoxy based overlay systems that use Mark 174 epoxy with a nominal maximum aggregate size (NMAS) of 0.187 inch (4.76 mm). Poly-Carb HFO systems are placed in a similar method as a chip seal.
- **SafeLane HFO systems** (product of Cargill, Inc.) are an epoxy overlay system with a nominal maximum aggregate size (NMAS) of 0.375 inch (9.5 mm). SafeLane HFO systems are placed in a similar method as a chip seal.
- **Transpo HFO systems** (product of Transpo Industries Inc.) are an epoxy overlay system using T-48 epoxy with a nominal maximum aggregate size (NMAS) of 0.187 inch (4.76 mm). Transpo HFO systems are placed in a similar method as a chip seal.
- **Novachip HFO system** (patented system of RoadScience LLC) are a thin-bonded open graded asphalt overlay with a 0.375 inch (9.5 mm) nominal maximum aggregate size (NMAS). Novachip HFO systems are placed in a similar method as a thin asphalt overlay. Novachip systems act more like thin asphalt overlays than like the epoxy based systems above.

In this study three different types of aggregates were used in Poly-Carb systems overlays including basalt, flint, and taconite. A description of the aggregates is found in the Aggregate section of Chapter 2.

Several different variables exist between the proprietary HFO systems including: (1) epoxy or asphalt overlay system type, (2) type and source of aggregate, (3) type of epoxy, (4) date of construction, and (5) contractor performing installation.

**Construction of an Epoxy High Friction Overlay System**

A short summary of construction processes observed during a site visit to Bridge 5718 in Sandstone, MN is provided for better insight on how an HFO system is placed on a bridge deck.

The system placed was a Poly-Carb system with basalt aggregate. Construction was performed by PCiRoads, a highway construction company based out of St. Michael, MN and was observed on September 4, 2013.

First the bridge deck is prepared by sandblasting the surface (Figure 1). This is done to provide a rough surface that will provide more surface area for the epoxy to adhere to.

Figure 1: Sandblasting a Concrete Deck to Prepare the Surface for an HFO System

The deck is cleaned after the sandblasting operation and the construction joints are prepared in order to create a level surface where adjacent lanes meet (Figure 2).

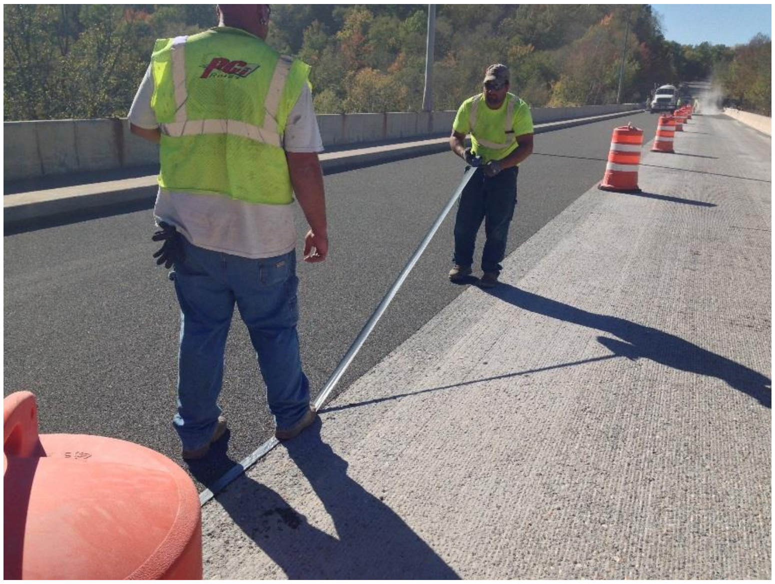

Figure 2: Construction Joints are Prepared

Epoxy is sprayed onto the roadway from a tanker truck and leveled using squeegee brooms (Figure 3).

Figure 3: Epoxy is Placed and Leveled

Aggregate is sprayed onto the fresh epoxy from a hopper. Once the aggregate is placed in the epoxy it is not adjusted (Figure 4).

Figure 4: Aggregate is sprayed onto the Layer of Epoxy

The entire construction process is conducted from a single truck keeping the work within a small proximity (Figure 5).

Figure 5: Construction Chain in a Small Proximity

Once the aggregate is placed, the system is left to cure (Figure 6). The epoxy used in HFO systems has a fast curing time and is able to take traffic loads within a few hours of installation completion.

Figure 6: Final Epoxy HFO System (Left Side: Epoxy HFO Lane, Right Side: Concrete Deck that Still Requires the Placement of an HFO Surface)

**Bridges with an HFO system installed**

In total, fourteen bridges in ten different locations were included in the study, however only eleven of the bridges are the main focus of research. The bridges are of various dimensions and carry different traffic loads at various speeds. These various characteristics of the bridges will be discussed further in subsequent sections. The eleven main bridges in this study were identified in the research work plan provided by MnDOT. Due to construction and maintenance schedules, three other bridges with epoxy systems outside the work plan's scope were visited. The observations conducted on these three bridges provided further data to compliment the study. Bridge 4190 (TH 55) and Bridge 9036 (Robert St) had traffic control in place to repair damaged sections of the epoxy systems. Bridge 5718 (TH 123) had an epoxy system placed in 2013 allowing for the observation of construction practices. Basic bridges properties are contained in Table 1. All bridges in this study have the HFO system placed on top of concrete decking.

Table 1: Property of Bridges with an HFO system

| Bridge Number | Location | Road-way | System Type | Year HFO Installed | Year Bridge Built | ADT | SR |
|---|---|---|---|---|---|---|---|
| 6347 | Osceola | TH 243 | Transpo | 2010 | 1953 | 7,600 | 65.6 (Adeq) |
| 9066 | Moorhead | I-94 WB | Poly-Carb | 2010 | 1958 | 31,000 | 91.6 (F.O) |
| 9067 | Moorhead | I-94 EB | Poly-Carb | 2010 | 1958 | 31,000 | 93.6 (Adeq) |
| 9823 | Atkinson | I-35 SB | Poly-Carb | 2009 | 1965 | 8,200 | 94.3 (Adeq) |
| 14809 | Barnesville | I-94 WB | SafeLane | 2007 | 1968 | 7,900 | 92.5 (Adeq) |
| 14810 | Barnesville | I-94 WB | SafeLane | 2007 | 1968 | 7,900 | 97.4 (Adeq) |
| 27019 | Otsego | TH 101 | Novachip | 2008 | 1993 | 19,750 | 98.5 (Adeq) |
| 27758 | Minneapolis | Penn Ave | Transpo | 2008 | 1986 | 10,300 | 92.2 (Adeq) |
| 69006 | Virginia | US 53 NB | Poly-Carb | 2009 | 1969 | 11,950 | 96.2 (Adeq) |
| 86013 | Otsego | CSAH 36 | Novachip | 2008 | 2006 | 7,700 | 99.4 (Adeq) |
| 86019 | Otsego | CSAH 36 | Novachip | 2008 | 2006 | 5,000 | 99.6 (Adeq) |

| Additional Bridges | | | | | | | |
|---|---|---|---|---|---|---|---|
| 4190 | St. Paul | TH 55 | Poly-Carb | 2011 | 1926 | 42,500 | 84.8 (Adeq) |
| 5718 | Sandstone | TH 123 | Poly-Carb | 2013 | 1948 | 2,050 | 62.3 (Adeq) |
| 9036 | St. Paul | Robert St | Poly-Carb | 2011 | 1926 | 19,000 | 74.0 (F.O) |

ADT = Average Daily Traffic
SR = Sufficiency Rating
Adeq = Adequate Structure
F.O. = Functionally Obsolete
Note: All information in this table was taken from the List of Minnesota Bridges as reported by the Minnesota Department of Transportation dated June, 23, 2009.

**Bridge Site Descriptions**

Different variables exist between the bridge locations. Some of these differences include: (1) the average daily traffic (ADT), (2) traffic speeds, (3) roadway alignment, (4) bridge structure, and (5) winter maintenance practices. The different lengths and widths of each bridge are listed in the Appendix and used for some later calculations. The following subsections provide a short description of each bridge. Multiple lanes were considered for bridges numbered between 9066 and 14810

***Bridge 4190***

Bridge 4190 is located in St. Paul, MN and is one of the three additional bridges in the study. The bridge is a four lane roadway with two outside shoulders and two four-foot interior shoulders with a concrete median. The bridge allows Trunk Highway (TH) 55 to cross over the Minnesota River. The eastbound lane was studied during the site visit.

***Bridge 5718***

Bridge 5718 is located in Sandstone, MN and is one of the three additional bridges in the study. It is a two lane bridge with one shoulder and a concrete sidewalk. The bridge allows Trunk Highway 123 to cross over the Kettle River.

***Bridge 6347***

Bridge 6347 is located at the Minnesota border crossing the St. Croix River to Osceola, WI. It is a two lane bridge with two four-foot shoulders. The bridge connects Minnesota and Wisconsin via Trunk Highway 243. An HFO system exists across the entire bridge surface. The eastbound lane was studied during the site visit; however due to inclement weather, minimal testing was conducted.

***Bridge 9036***

Bridge 9036 is a four lane bridge that allows Robert Street to cross over the Mississippi River. It is one of the three additional bridges in the study. The bridge is located in downtown St. Paul, MN. The northbound lane was studied during the site visit.

***Bridge 9066***

Bridge 9066 is located in Moorhead, MN. It is a three lane bridge with two shoulders that allows westbound Interstate 94 to cross over the Red River.

***Bridge 9067***

Bridge 9067 is located in Moorhead, MN. It is a three lane bridge with two shoulders that allows westbound Interstate 94 to cross over the Red River.

***Bridge 9823***

Bridge 9823 is located on southbound Interstate 35 near Atkinson, MN. The bridge crosses over a county road (Old Highway 61), and has two lanes and a shoulder. The bridge has a curve in its alignment as well as a negative grade.

***Bridge 14809***

Bridge 14809 is located in Barnesville, MN. The bridge has two lanes and a shoulder. It is a railroad overpass allowing westbound Interstate 94 to cross over the tracks. The location of the bridge is completely surrounded by open farm land.

***Bridge 14810***

Bridge 14810 is located in Barnesville, MN. The bridge has two lanes and a shoulder. It is a two lane railroad overpass allowing eastbound Interstate 94 to cross over the tracks. The location of the bridge is completely surrounded by open farm land.

***Bridge 27019***

Bridge 27019 is located in Otsego, MN. The bridge allows the two southbound lanes of Trunk Highway 101 to cross over the Crow River.

***Bridge 27758***

Bridge 27758 is located in Minneapolis, MN. The bridge allows a local road, Penn Ave, to cross over Interstate 394 just west of downtown Minneapolis. The bridge has four lanes (two driving lanes and two turn lanes) with a curb median separating the two directions of traffic. The overlay system only covers the northbound lanes.

***Bridge 69006***

Bridge 69006 is located on US Highway 53 headed northbound in Virginia, MN. The two lane bridge crosses over a local road and has a curve in its alignment as well as a negative grade.

***Bridge 86013***

Bridge 86013 allows CSAH 36 to cross over TH 101 in Otsego, MN. The bridge crosses just north of Bridge 27019's location, has six lanes (four driving lanes and two turn lanes) and two shoulders. The HFO system is only placed on the westbound lanes.

***Bridge 86019***

Bridge 86019 allows CSAH 36 to connect with TH 101 in Otsego, MN. The bridge crosses over the Crow River and merges with TH 101 just south of Bridge 27019's location. The bridge has one lane and two four-foot shoulders.

# Chapter 2: Literature Review

Literature from the different proprietary companies market the HFO systems to provide three features that improve the infrastructure they are placed on: (1) increased friction, (2) infrastructure protection, and (3) crash reduction. These three features are all provided by HFO systems to keep drivers safer (Cargill, 2014). Previous research has been conducted in order to evaluate if HFO systems provide the benefits they claim, and such evaluation is also the main objective of this research. This study however varies from previous research in two ways: the climate the research is conducted in and the scale of the project. The first difference is the climate. Harsh winter climates exist in Minnesota and often these climates last four to six months out of the year. Not only do the winter elements take a toll on infrastructure, but so do the winter maintenance practices. A large portion of previous research has taken place in more mild climates that does not encounter such extreme conditions and heavy use of snow plows. The second variation from previous research is that this research has a larger scope. Previous studies have had a similar time frame, number of bridges investigated, number of proprietary products investigated, number of tests conducted, or scale of analytical comparison for crash data, but each previous study has only focused on a few of these variables.

This study is comprehensive and provides a complete look at different HFO systems and the variables that affect the life of the system. The research was conducted in an environment that truly tests the HFO system's winter performance.

## Minnesota Climate

Minnesota, part of the Upper Midwest Region, is located in the northern part of the continental United States. The state has a continental climate, which is a climate with large annual variations in temperature. With a record minimum temperature of -60 degrees Fahrenheit [-51 Celsius] in 1996, and record maximum temperature of 114 degrees Fahrenheit [46 Celsius] in 1936 (MnDNR, 2014a), roadways in Minnesota are exposed to a variety of conditions. The mean annual temperature ranges from 36 degrees

Fahrenheit [2 Celsius] in the northern part of the state, to 49 degrees Fahrenheit [9 Celsius] in the south. Average annual precipitation in the state ranges from 19 inches [48 cm] in the north to 39 inches [99 cm] in the south. Seasonal snowfall averages from 70 inches [178 cm] in the northeast to 40 inches [102 cm] in the south. Heavy snowfalls of over four inches [10 cm] are common place between the months of November and April (MnDNR, 2014a).

Snowfall and cold temperatures during these months can create safety concerns on roadways and especially bridges as surfaces become wet and/or frozen. It is because of these conditions that authorities are constantly researching ways to improve the winter time safety of roadways, and the reason high friction overlays are being looked at by the state's Department of Transportation. The bridges in this study can be broken into three distinct regions of the state, northwest, northeast, and central Minnesota. The bridges located in each region are listed below in Table 2. Climatic information regarding these three regions can be found below in Table 3. The information in this table is from the Minnesota Department of Natural Resources and was collected by major airport weather stations in Fargo, ND (northwest region), Duluth (northeast region), and Minneapolis/St. Paul (central region) (MnDNR, 2014b). The main weather concerns occur during the winter months which for this research is considered between the months of October and March.

Table 2: Bridges Located in Each of the Three Regions

| Region | Bridges Located in Region |
|---|---|
| Northwest | 9066, 9067, 14809, and 14810 |
| Northeast | 9823, 69006 |
| Central | 6347, 4190, 5718, 9036, 27019, 27758, 86013, and 86019 |

Table 3: Average Winter Temperatures, Precipitation, and Snowfall for the Different Regions of Minnesota (October-March)

| Region | Northwest | Northeast | Central |
|---|---|---|---|
| Weather Station Location | Fargo | Duluth | Minneapolis/ St. Paul |
| Mean Daily Max (°F) [°C] | 31.0 [-0.5] | 30.3 [-0.9] | 35.4 [1.9] |
| Mean Daily Min (°F) [°C] | 12.8 [-10.7] | 13.4 [-10.3] | 19.1 [-7.2] |
| Normal Precipitation (inches) [cm] | 1.0 [2.5] | 1.5 [3.8] | 1.5 [3.8] |
| Normal Snowfall per Month (inches) [cm] | 7.4 [18.8] | 12.7 [32.3] | 8.7 [22.1] |

This cold region provides an excellent location to test and observe if HFO systems provide their design benefits during winter conditions.

## Aggregates

In this study three different types of aggregates were used in Poly-Carb overlays. The three different aggregates are basalt, flint, and taconite. The basalt aggregate is a crushed (igneous) river rock from Washington. The flint aggregate was quarried in Oklahoma, and the taconite aggregate is a byproduct of mining operations in northern Minnesota. A more geological description of taconite is that "taconite is an iron-bearing sedimentary rock composed of alternating chert and slate units of varying thickness" (Zanko, Fosnacht, & Hopstock, 2009). SafeLane uses a proprietary aggregate that consists of Limestone (Cargill, 2014).

Since the friction values of the HFO systems come from the macrotexture and microtexture of the aggregate, its strength and integrity are important. The soundness and durability of aggregates are especially important when used in cold climate regions (Mamlouk, 2011). Cold climate regions require aggregates to have a high resistance of degradation due to abrasion and freeze thaw damage. This resistance to degradation is especially important in HFO systems, as opposed to traditional concrete pavements, because the aggregates are directly exposed to loads, abrasion, and weather. The abrasion resistance of aggregates can be tested through several tests including the use of a Los Angeles abrasion machine (ASTM C131, 2014) and in a freeze-thaw chamber in accordance to AASHTO T103-08.

The shape of aggregate particles can help improve the resistance to abrasion and friction values of aggregates. Aggregate particles with multiple faces have sharper corners compared to particles with smooth features. HFO systems need aggregates with these rough features to increase the overall surface friction as well as improving the bonding with an epoxy or asphalt binder (Mamlouk, 2011). Flat and elongated aggregates have a dimension that is significantly smaller compared to the other dimensions of the particle. It is desirable to have aggregates with even dimensions because thin aggregates are more likely to break along a plane parallel to its smallest dimension. Aggregates of cubical shape will create an even friction surface once placed into an epoxy or asphalt matrix. The shape of aggregates in HFO systems are tested through ASTM D5821-13 (percent of fractured faces) and ASTM D4791 (flat and elongated particles).

**Macrotexture and Microtexture of Aggregates**

The macro- and microtexture of aggregates are an important aspect of HFO systems. These two types of texture contribute to the higher friction found in such systems. Macrotexture is the texture supplied by the shape of an aggregate. Aggregates with sharp edges and crushed faces have a higher macrotexture compared to rounded aggregates. Microtexture is the texture found on the surface of an aggregate. High microtexture indicates that an aggregate has a rough or unpolished surface. An illustration of these two types of textures relative to a roadway is shown in Figure 7.

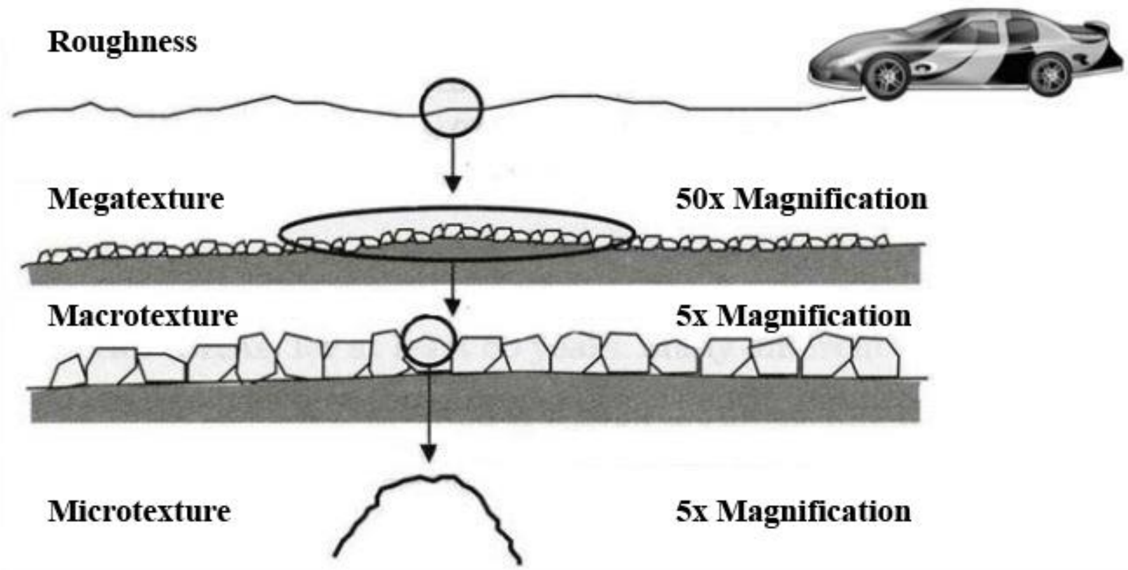


Figure 7: Macrotexture and Microtexture of Aggregates (Source: Nelson, 2011)

Both types of texture are important for the high friction in HFO systems. Distresses that reduce these textures over time are discussed later in Chapter 4.

## HFO Systems in Minnesota

MnDOT has previously performed a study evaluating HFO systems for crash reduction on bridges. The three year study was conducted from 2006 to 2009 and looked at three different locations where SafeLane HFO systems were installed. The locations were Alexandria, Barnesville (the SafeLane bridges in this study), and Bemidji. The researchers also observed the installation of a SafeLane HFO system in Hibbing. The research looked at several aspects of the system including: (1) bonding to the deck (2) chloride intrusion prevention, (3) chloride retention on surface, (4) traction measurements, and (5) accident reduction.

The study came to several conclusions including that when properly placed, a strong adherent bond between the HFO system and the bridge deck is developed. This bond provides excellent protection against the intrusion of chloride, and it is inferred that this protection will help decrease the corrosion of reinforcing steel in the underlying bridge deck over its service life. The overlays provided an improved traction and retained deicing chemicals on the surface to help prevent the buildup of ice. The research concluded that the HFO systems reduce accident rates both directly (improved traction) and indirectly (deicing chemical retention).

The study also concluded that “significant, rapid wear” occurs primarily due to snow plow shearing forces (abrasion) as well as from traffic loads in the wheel path. One of the bridges in the study (Hibbing, MN) had a skid number (SN) of 80 when installed in 2006. By 2009 the SN of the HFO system had dropped to 35 (Evans, 2010). A description of skid numbers is later in this chapter. Due to heavy abrasion, the service life of an HFO system in Minnesota is estimated at 3.5 to 5 years.

Finally the report concluded that the study was too short and a more comprehensive study that encompasses other proprietary HFO systems should be conducted (Evans, 2010).

## HFO Systems in Other Sates

Previous research of HFO systems has been conducted in other states including, Missouri (Harper, 2007), Colorado (Young, Durham, & Bindel, 2012), Vermont (Tremblay, 2013), Washington (Anderson et al., 2012), and Wisconsin (Izeppi, 2010). When discussing HFO systems with a Transpo representative during a site visit, he mentioned that HFO systems were becoming especially popular in Kansas and Missouri. Previous research can be broken down into two different tasks, field observation and crash analysis.

### Field Observations

Previous research of HFO systems has been conducted to evaluate the performance of such systems in the field. The two main questions research has looked to answer are: (1) how do HFO systems perform in the field? (2) What is the service life of an HFO system? The most extensive research is in the field performance of HFO systems. Several studies test the mean texture depth, bond strength, chloride penetration, and skid resistance (Young, Durham, & Bindel, 2012). Each of these test methods help to evaluate different aspects of HFO systems in order to answer the previous questions about HFO systems. Below in Table 4 the different test methods used by previous research are listed.

Each test is used to help evaluate one or more of the three benefits that HFO systems provide. It should be noted that the permeability test was added to this table because of its use in this study. A further description of how tests are conducted is provided later in Chapter 4.

Table 4: Test Methods and the High Friction Overlay Benefit Evaluated

| Test Method | HFO System Benefit |
|---|---|
| Mean Texture Depth | Higher Friction |
| Bond Strength | Infrastructure Protection |
| Permeability Test | Infrastructure Protection |
| Chloride Ion Penetration | Infrastructure Protection |
| Skid Resistance | Higher Friction and Crash Reduction |

A declination in the mean texture depth and skid resistance over time are both common results found in previous research. The skid resistance is often monitored to observe how long systems keep skid numbers over 50 (Tremblay, 2013). Skid numbers and their use in research is expanded upon in its own section later in this literature review.

Along with the performance of the aforementioned tests, another important aspect to observe during site visits include distresses that occur in HFO systems.

Distresses observed in previous studies that can lead to the failure of an overlay system include aggregate raveling, delamination, or polishing of the aggregate (Izeppi, 2010). Aggregate raveling and polishing reduce the friction values of HFO systems while delamination eliminates the infrastructure protection benefit (Harper, 2007). Research conducted in Missouri looked into the cause of an HFO system delamination, which is the separation of the HFO system from the bridge deck surface. The research noted that data sheets of the Sikadur 22, Lo-Mod (Epoxy System) states that a concrete bridge deck should be free of moisture before installation, however some contractors consider "free of moisture" to be no standing water on the bridge deck. The research recommends the use of ASTM D 4263 "Standard Test Method for Indicating Moisture in Concrete by the

Plastic Sheet Method” in order to ascertain whether there is any moisture in the bridge deck (Harper, 2007).

The testing and observations are conducted in order to evaluate the performance of HFO systems and to assess their life span (Izeppi, 2010). This life span can be shortened even before traffic is placed on an HFO system due incorrect application and distribution of epoxy and aggregates during construction. Often it is difficult to apply the correct amounts of material and inexperienced personnel may be unable to apply HFO systems to bridge decks uniformly (Anderson, 2012). The multitude of variables that can lead to a shorter life span in HFO systems highlights the need for comprehensive field observations and evaluation.

**Crash Analysis**

Previous research has used crash analysis as a means to justify the use of an HFO system. Research has been conducted on HFO systems in order to answer two questions regarding crashes on bridges: (1) does the higher friction reduce the amount of crashes or the severity of crashes on a bridge? (2) If so, do HFO systems provide an economical benefit through this reduction in crashes?

Several studies have investigated the crash rates both before and after the installation of HFO systems. In Wisconsin the study looked at four different bridges and studied a six year span of crash data including three years of data before and three years after the installation (Izeppi, 2010). The study notes a decrease in crash rates during the post-installation years. A study conducted in Colorado investigated HFO systems. Part of the study looked at accident history with results that were inconclusive if the number of weather related accidents was reduced after the installation of an HFO system (Young, Durham, & Bindel, 2012).

An economical benefit is a second way to justify the use of HFO systems on bridge decks. Does the reduction in costs due to a reduction in crashes have a greater monetary value than the cost of an HFO system installation and maintenance? The cost of accidents vary from different sources, however MnDOT has a typical monetary value associated with each crash severity class (Izeppi, 2010). These values are shown below in Table 5. Using such values, a benefit-cost analysis can be conducted.

Table 5: Associated Costs with Different Crash Types (MnDOT)

| | |
|---|---|
| No Apparent Injury | $4,400 |
| Possible Injury | $30,000 |
| Non-Incapacitating Injury | $61,000 |
| Incapacitating Injury | $280,000 |
| Killed | $3,600,000 |

The previously mentioned Wisconsin study performed a benefit-cost (B/C) analysis on several bridges using the monetary values in Table 5. The ratio compares the reduced cost of crashes against the cost of installing an HFO system. A value over 1.0 means that the benefits outweighed the costs of installation. The B/C ratios in the Wisconsin study ranged from 0.46 to 8.45 with an overall B/C ratio of 1.86 (Izeppi, 2010).

## Skid Numbers

This subsection on skid number measurements provides an overview of the current practice used to measure the surface friction properties of roadway surfaces. This section used the measured skid numbers for HFO systems in this research to compliment the literature review of skid numbers. The results are used later in the paper to make comparisons with crash reduction efficiencies of various HFO types.

Skid numbers are used to compare the friction values of different roadway surfaces, and current research uses skid numbers as indicators for roadway safety. A skid number (SN) is a calculated number that allows the comparison of frictional values for different roadways (Watson, 2011). Equipment used to determine these values and their test procedures are further discussed in Chapter 4. An SN value is the ratio of the horizontal force to the vertical load while an object moves over a surface multiplied by 100. Water is sprayed onto the roadway in front of the test tire in order to simulate the “worst case” scenario of a wet road. MnDOT collects skid numbers for roadways using a KJ Law (Dynatest) Skid Trailer. Ribbed tires, which have a tread design, and smooth tires were used for this testing. Ribbed tires are more sensitive to the microtexture of a surface while smooth tires are more sensitive to the macrotexture (Watson, 2011). A certain skid number however does not necessary correlate to a safe or unsafe road condition (FHWA, 2010).

Although no relationship currently exists, recommended threshold values were determined based on research sponsored by the Texas Department of Transportation. (Long, 2014). Of these recommendations two important thresholds exist. The first is a range of SN where roadways are “potential action segments”. These segments may require testing or inspection to determine if treatment is required to improve the roadway’s safety. The second threshold is where an increased SN values has little effect on reducing crash rates (Long, 2014). According to the Texas research, for the type of highway segments found in the following research conducted, an SN value between 10 and 50 are in the “potential action segment” threshold. SN values over 50 have little effect in reducing crash rates by increasing the SN. Furthermore some research cited by this Texas study has concluded that relationships between skid resistance values and crash rates are not significant, especially on freeways or higher speed roadways (Long, 2014).

The SN values for nine bridges in this study are shown below in Table 6. Testing was conducted from 2011 to 2013 (Three to Five years after the HFO installation). The SN values shown in Table 6 show that several HFO systems are still above 50 several years after installation while others have fallen below 50 into the “potential action segment” threshold.

Table 6: Average Skid Numbers for HFO Systems on Bridges (Data from 2011 to 2013)

| Bridge | 6347 | 9066 | 9067 | 9823 | 14809 | 14810 | 27019 | 69006 | 86013 |
|---|---|---|---|---|---|---|---|---|---|
| SN | 58.3 | 41.7 | 44.0 | 54.1 | 36.5 | 35.1 | 54.5 | 58.8 | 56.4 |
| Year of HFO Installation | 2010 | 2010 | 2010 | 2009 | 2007 | 2007 | 2008 | 2009 | 2008 |

As roadways age their SN values deteriorate. Abrasion from traffic and snowplows facilitate the deterioration of SN values. The heavy use of snowplows in Minnesota due to the state's harsh winter climate creates higher abrasion rates compared to mild-climate regions. For a comparison, the SN values of a Portland cement concrete (PCC) pavement (traditional bridge deck overlay in Minnesota) are as follows. Immediately after construction SN values range from 55-60 (ribbed tires) or 45-50 (smooth tires). Five years after construction the skid numbers drop to 45-50 for ribbed tires and to 25-30 for smooth tires (Nelson, 2011).

Since a minimum skid resistance value does not exist for crash reduction, it is possible to improve skid resistance of a bridge deck without actually improving the safety of the bridge. Using crash data provided by MnDOT from the public safety database, this research will analysis and evaluate the effectiveness of HFO systems in reducing crashes on bridges.

## Building Off of Previous Research

Previous research in other states have conducted field tests very similar to the tests in this study including: (1) mean texture depth, (2) bond strength, (3) skid resistance, and (4) chloride penetration. While previous research has been conducted on the performance of HFO systems, and several assessments of crashes on HFO systems exist, a robust crash analysis has not been conducted. Minimal research has taken a significant look at how effective systems are in reducing crashes. This research will look at a decade of crash data in order to ascertain the effectiveness of HFO systems in terms of crash reduction.

In the study conducted by MnDOT (Evans, 2010), the recommendation to create a more comprehensive study was the motivation for this current research. This study looks at four different types of HFO systems instead of one. The field testing will compare the wheel to non-wheel path areas, and finally an analysis of ten years of crash data will be conducted. The hope is that this study will be able to either confirm or disprove the conclusions of the previous research as well as conclude if the use of HFO systems in Minnesota is beneficial.

# Chapter 3: Comprehensive Aggregate Testing

The role of aggregates in these systems is critical to ensure that necessary friction is maintained over the service life of the wear courses. The main areas of concern in context of the performance of wear courses include: (1) retention of aggregate particles, (2) degradation and polishing of aggregate particles under the actions of traffic and snow plows, and (3) durability of aggregates under repeated freezing and thawing conditions. A number of laboratory tests can be conducted to get insight on the aforementioned areas of concern with respect to aggregates.

A comparison of several different aggregate sources that were implemented in variations of these systems follows, including the five aggregate types used in Poly-Carb, Transpo, and SafeLane systems that were investigated. Table 7 provides details of the five aggregate samples that were chosen to undergo the testing. These products were chosen on the basis of systems actively in service as well as availability of sample material.

Table 7: Aggregate Information

| Aggregate | Bridge | Roadway | Location | Source |
|---|---|---|---|---|
| Basalt | 4190 | Hwy 55 (EB) | St. Paul | Washington Rock Quarries (Pierce, WA) |
| | 9036 | Robert St | St. Paul | |
| | 9823 | I-35 (SB) | Atkinson | |
| Flint | 6347 | TH 243 | Otsego | Flint Rock Products (Picher, OK) |
| | 9066 | I-94 (WB) | Moorhead | |
| | 9067 | I-94 (EB) | Moorhead | |
| Taconite A | 69006 | TH 53 (NB) | Virginia | United Taconite (Eveleth, MN) |
| Taconite B | N/A | N/A | N/A | ArcelorMittal Minorca Mines (Virginia, MN) |
| SafeLane | 14809 | I-94 (WB) | Barnesville | N/A |
| | 14810 | I-94 (EB) | Barnesville | |

A couple additional notes about the processing of the aggregate include:

- Basalt consists of river rock that was crushed to specifications for an HFO system
- Flint: Both Poly-Carb and Transpo products use flint from Oklahoma
- Taconite-A was prepped at the Ulland Brothers' Kinmount location in Orr, MN
- SafeLane is a proprietary aggregate by Cargill and at this time Cargill will not disclose the source location
- The aggregate source used for the Novachip systems in this study is unknown
- Trap Rock was used for Bridge 27758 (Penn Ave, Transpo) however the aggregate was not provided for this portion of the study.

Pictures of all five aggregates with a reference scale are found below in Figure 8 through Figure 12.


Figure 8: Basalt Aggregate


Figure 9: Flint Aggregate


Figure 10: Taconite-A Aggregate

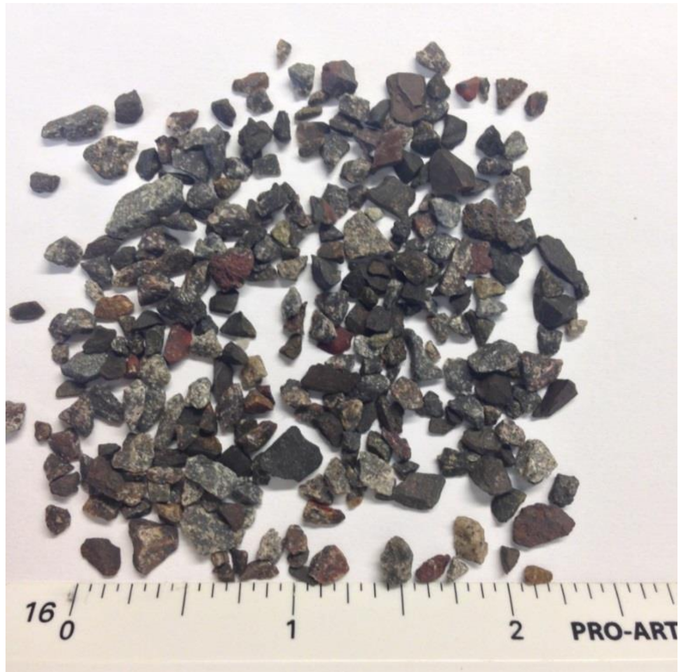


Figure 11: Taconite-B Aggregate


Figure 12: SafeLane Aggregate

## Testing Procedures and Methodology

Testing was completed on aggregate samples in order to acquire material properties. The desired properties, tests, and their corresponding ASTM or AASHTO specification designations are in Table 8. Testing was conducted at the University of Minnesota Duluth civil engineering laboratory.

Table 8: Properties and Preliminary Test Method

| **Property** | **Test Method** |
|---|---|
| Density, Specific Gravity, and Absorption | ASTM C127-12 – Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Coarse Aggregate<br><br>ASTM C128-12 – Standard Test Method for Density, Relative Density (Specific Gravity), and Absorption of Fine Aggregate |
| Resistance to Degradation | ASTM C 131-14 – Standard Test Method for Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in the Los Angeles Machine |
| Uncompacted Void Content | ASTM C 1252-06 – Standard Test Methods for Uncompacted Void Content of Fine Aggregate (as Influenced by Particle Shape, Surface Texture, and Grading |
| Flat/Elongated Particles | ASTM D4791-10 – Standard Test Methods for Flat Particles, Elongated Particles, or Flat and Elongated Particles in Coarse Aggregate |
| Percent of Fractured Particles | ASTM D5821-13 – Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate |
| Soundness of Aggregates | AASHTO T 103-08 – Standard Method of Test for Soundness of Aggregates by Freezing and Thawing |
| Sieve Analysis | ASTM C136-06 – Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates |

## Description of Standard Tests

The following sections provide a brief overview for each of the tests completed during this task.

### Density, Specific Gravity & Absorption (Coarse Aggregate)

The ASTM C127-04 and ASTM C128-07 find the saturated-surface dry and dried mass values of an aggregate in order to calculate density, specific gravity, and absorption of coarse aggregate using gravimetric relationships. High specific gravity usually correlates with high toughness and strength.  High absorption is an indicator of potentially low durability which can lead to a shorter life span on an aggregate in an HFO system.

### Resistance to Degradation

ASTM C131-06 uses a Los Angeles abrasion machine to measure the degradation of mineral aggregates due to abrasion, impact, and grinding in a rotating steel drum. The Los Angeles Machine is pictured in Figure 13. This provides an indication of aggregate toughness and potential for breakdown under abrasive actions such as traffic and snow plows.

Testing in this study had one deviation from the ASTM standards. The standard requires the aggregate retained on the #12 sieve to be washed after rotating in the steel drum. For this testing aggregate retained on or above the #10 sieve was used because a #12 sieve was unavailable at the time of testing.

Figure 13: Los Angeles Machine

**Uncompacted Void Content**

The uncompacted void content can be measured using a funnel and calibrated cylinder (Figure 14). The uncompacted void content is the difference between the volume of the standard cylinder and the absolute volume of the fine aggregate collected in the cylinder. Uncompacted voids provide an indication of the angularity of aggregates and hence provide an approximation of the friction that will be offered through macrotexture. The higher the uncompacted void content, the larger the spacing is between aggregates which allows for a well-drained system.

Figure 14: Aggregate Sample Falling into Calibrated Cylinder

**Flat/Elongated Particles**

Individual aggregate particles are measured to determine the ratios of width to thickness, length to width, or length to thickness. Figure 15 displays the equipment used to measure aggregate particles. Ratios of 2:1, 3:1, and 5:1 were measured. Flat and elongated particles are susceptible to break down due to a thin dimension in the individual particles. Higher percentages of flat and elongated particles in an aggregate stockpile may signify the potential for low durability and a shorter life span of the HFO system.

Figure 15: Flat/Elongation Measuring Equipment with Sorted Aggregate

**Percent of Fractured Particles**

ASTM D5821-01 has individual particles of aggregate surveyed to find the percentage of fractured particles in an aggregate. Fractured particles are important to provide high friction surfaces as well as provide surface area for epoxy to properly bond.

**Soundness of Aggregates**

The resistance to disintegration of aggregates by freezing and thawing is found by the AASHTO T 103-08 specification. This testing subjects a sample of aggregate particles to freeze thaw cycles in a controlled chamber. Weather in MN is conducive to repeated freezing and thawing, thus it is important to know if an aggregate has sufficient durability for such conditions. More resistant aggregates are desirable for longer lasting systems and maintaining a high friction surface.

**Sieve Analysis**

A sieve analysis was run on all five aggregate samples to determine the relative proportions of different grain sizes. A sieve analysis is done to evaluate an aggregate's gradation to ensure: (1) a limited amount of fines to allow proper bonding between epoxy layers and the aggregate particles, (2) an open graded nature to provide good macrotexture, drainage, and friction, and (3) to compare to MnDOT chip seal requirements since epoxy systems are constructed in a similar fashion.

## Results

Results found from the aforementioned standard test methods are provided below with additional equations, data, and calculations in the Appendix.

### Density, Specific Gravity & Absorption

Density (Table 9), Specific Gravity (Table 10) and Absorption (Table 11) were found using gravimetric measurements in accordance to ASTM standards.

Table 9: Density

| Aggregate | Oven Dry Density ($kg/m^3$) | SSD Density ($kg/m^3$) | Apparent Density ($kg/m^3$) |
|---|---|---|---|
| Basalt | 2,455 | 2,561 | 2,746 |
| Flint | 2,223 | 2,407 | 2,725 |
| Taconite A | 2,695 | 2,841 | 3,159 |
| Taconite B | 2,642 | 2,776 | 3,051 |
| SafeLane | 2,482 | 2,618 | 2,873 |

Table 10: Specific Gravity

| **Aggregate** | **Oven Dry Relative Density (Specific Gravity)** | **SSD Relative Density (Specific Gravity)** | **Apparent Relative Density** |
|---|---|---|---|
| Basalt | 2.46 | 2.57 | 2.75 |
| Flint | 2.23 | 2.41 | 2.73 |
| Taconite A | 2.70 | 2.85 | 3.17 |
| Taconite B | 2.65 | 2.78 | 3.06 |
| SafeLane | 2.49 | 2.62 | 2.88 |

Table 11: Absorption

| **Aggregate** | **Absorption (%)** |
|---|---|
| Basalt | 1.36 |
| Flint | 1.34 |
| Taconite A | 1.18 |
| Taconite B | 1.06 |
| SafeLane | 1.45 |

Taconite-A and Taconite-B respectively are the two densest aggregate samples tested. SafeLane and Basalt are of similar density. Flint was the lightest of the five aggregates. SafeLane had the highest abortion percentage compared to the other samples; the two Taconite samples had the lowest absorption percentage.

**Resistance to Degradation**

Resistance to degradation is determined by measuring the percent of mass lost due to abrasion and impact. Table 12 provides the results for the five different samples.

Table 12: Loss by Abrasion in LA Abrasion Machine

| Aggregate | Loss by Abrasion and Impact |
|---|---|
| Basalt | 3.1% |
| Flint | 3.0% |
| Taconite A | 3.2% |
| Taconite B | 4.0% |
| SafeLane | 4.6% |

All of the samples displayed similar results and performed well in the abrasion test. In comparison, according to the Pavement Interactive website, a typical loss for hard igneous rocks is around 10% (Pavia Systems Inc., 2012). Both Taconite samples and SafeLane had higher relative loss due to abrasion. These were also the aggregates with the highest specific gravity. There may be a correlation with the higher mass particles which create a higher impact with one another when tumbling in the LA abrasion drum.

**Uncompacted Void Content**

The uncompacted voids for the five aggregate samples is provided in Table 13. The calculation process can be found in the Appendix.

Table 13: Uncompacted Void Content Data

| Aggregate | Basalt | Flint | Taconite A | Taconite B | SafeLane |
|---|---|---|---|---|---|
| Uncompacted Voids (%) | 38.60% | 36.40% | 39.40% | 37.40% | 42.30% |

The sample with the largest uncompacted void content was SafeLane. The Poly-Carb aggregate samples all had similar uncompacted void percentages. The four Poly-Carb samples had a 3% variation while the SafeLane aggregate was 3% higher than the Poly Carb sample with the largest uncompacted void content. Since SafeLane aggregates are larger in size compared to the Poly-Carb aggregates, logically more space will exist in between particles.

**Flat/Elongated Particles**

This test was performed on particles that were retained on the #4 sieve. Particles were characterized by being, flat, elongated, flat and elongated, or no characteristics. The ratios tested were 2:1, 3:1, and 5:1. The percentages of different shaped particles for each sample are plotted in Figure 17 through Figure 19. A flat particle is a particle that has a width to thickness ratio larger than the specified ratio. An elongated particle is a particle that has a length to thickness ratio larger than the specified ratio. Flat and elongated particles have both ratios larger than the specified ration. A visual of the particle dimensions is shown below in Figure 16.

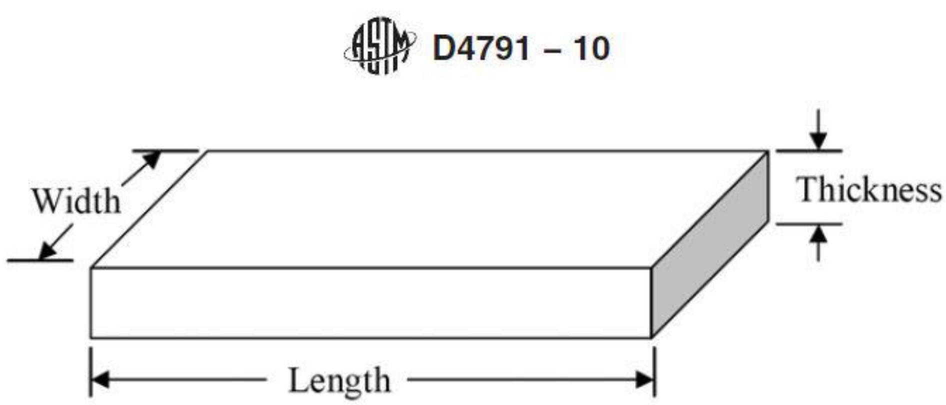


Figure 16: Particle Dimensions per ASTM D4791-10 (Source: ASTM D4791-10, 2010)

For the 2:1 ratio a particle that is twice as wide as it is thick is considered flat. A particle that is twice as long as it is thick is elongated. A particle matching both requirements is a flat and elongated particle. It should be noted that the Flint sample did not have enough particles retained on the #4 sieve to perform this test.

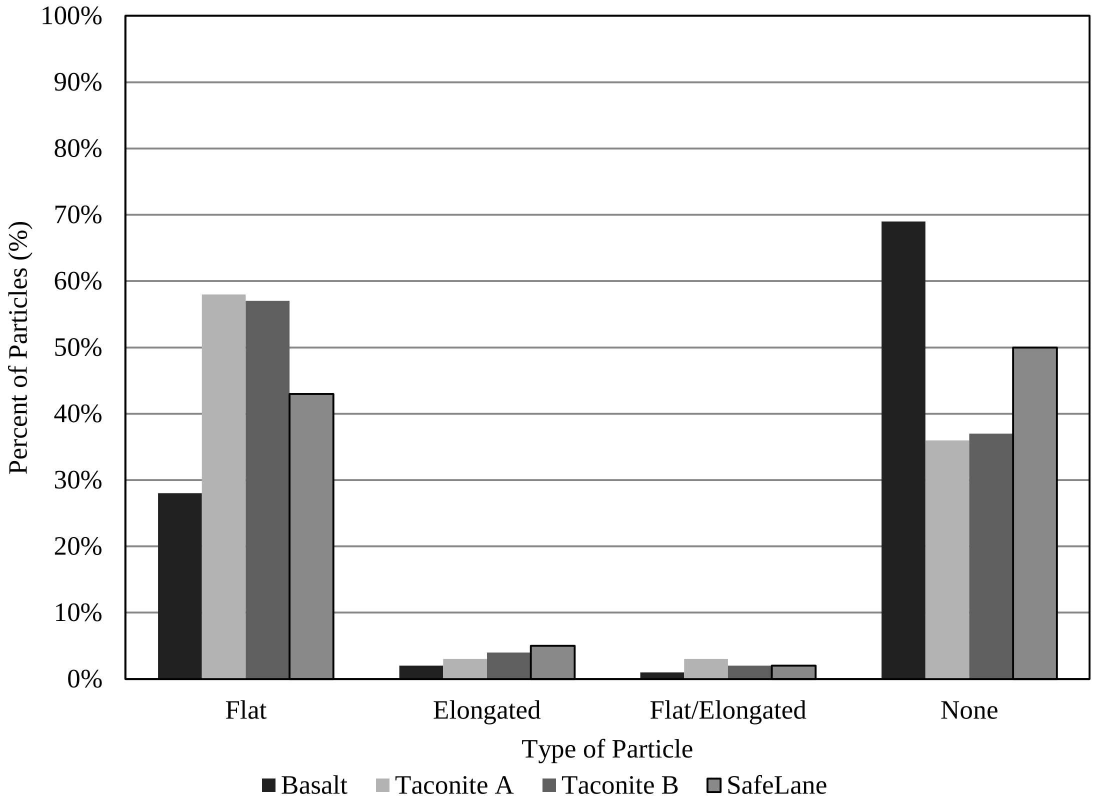


Figure 17: Flat and Elongated Particles (2:1 Ratio)

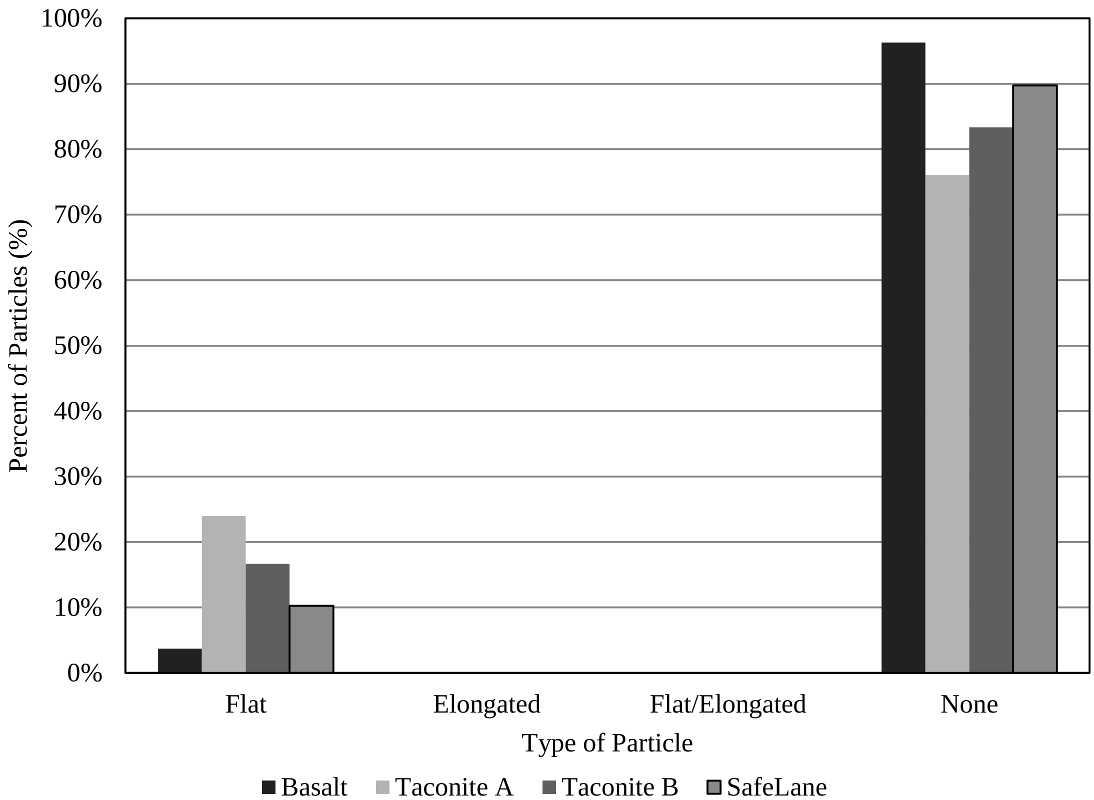


Figure 18: Flat and Elongated Particles (3:1 Ratio)

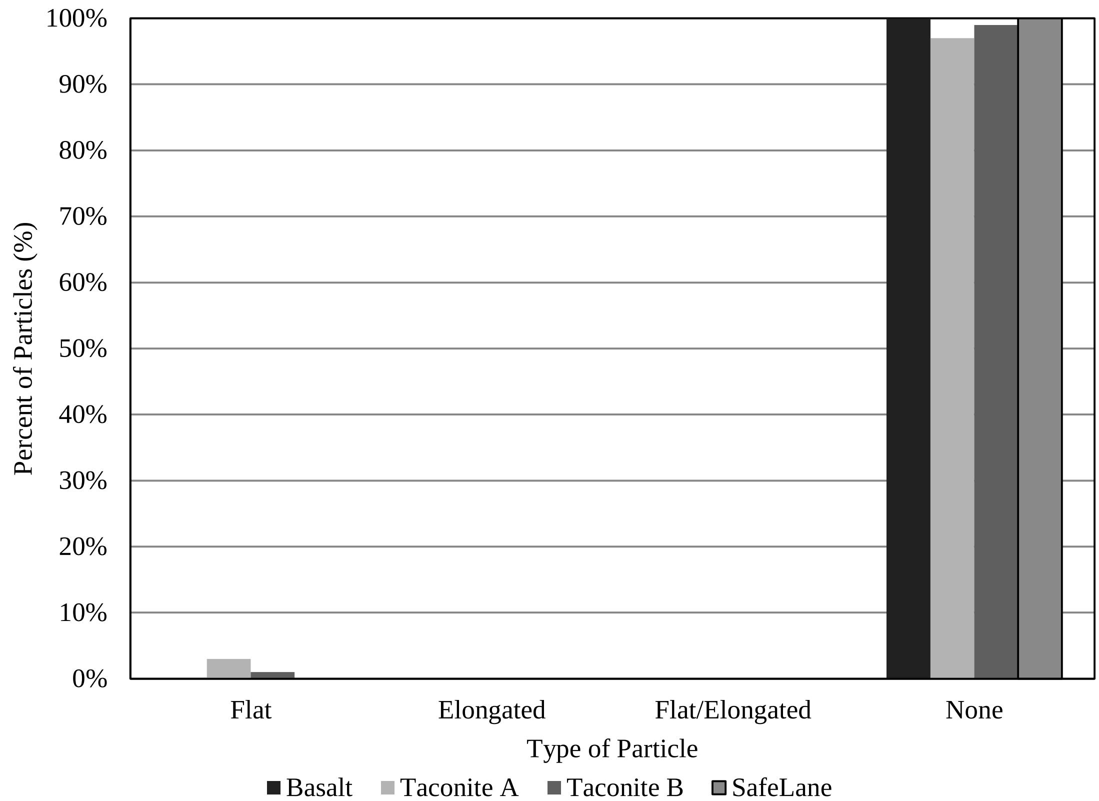


Figure 19: Flat and Elongated Particles (5:1 Ratio)

All four of the samples tested have a significant portion of flat particles in the 2:1 ratio. However this falls off quickly as the ratio is increased. Taconite "A" contains the most flat particles across all three ratios with Taconite "B" following closely behind. Basalt has the lowest percentage of flat and elongated particles which may contribute to its low mass loss due to abrasion. It should be noted that although the Flint aggregate had particle sizes too small to run this test on. Based on visual inspection, the aggregate sample contains more flat and elongated particles compared to the other samples.

**Percent of Fractured Particles**

After examination of the five crushed aggregate sources, it was determined that 100% of the particles in all five sources have at least 2 fractured faces. This is a reasonable characteristic because most of the aggregate samples are mechanically crushed in order to obtain the desired aggregate sizes.

### Soundness of Aggregates

The sieve sizes used during this testing were based off of the Sieve Analysis. Sieve sizes with minor amounts of particles retained after freeze-thaw conditioning were excluded from testing. This is shown in Figure 20 with the text “No Data”. For each aggregate sample, the different sieve sizes were combined to create a total mass. The total mass lost after being subjected to freeze-thaw conditions was normalized with the total sample size for each aggregate allowing for a comparison between the different aggregates (Figure 21).

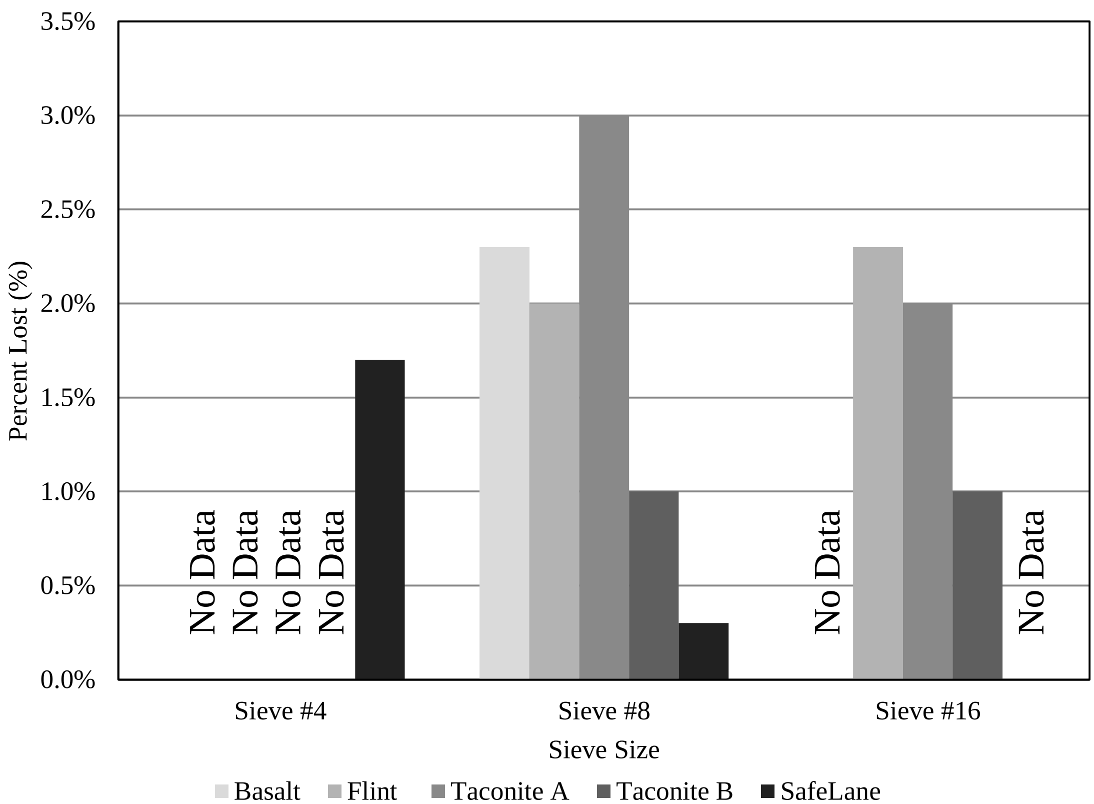


Figure 20: Percent Lost Due to Freeze Thaw

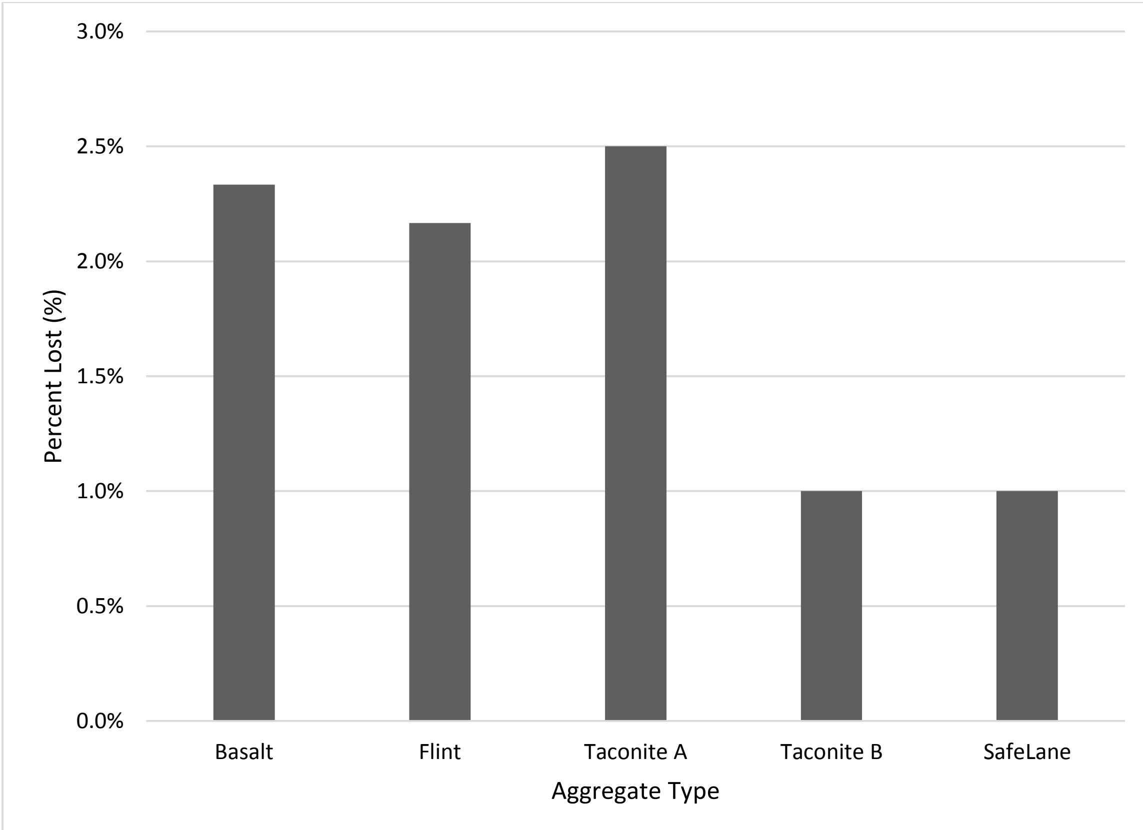


Figure 21: Percent Lost Due to Freeze Thaw (Normalized to Sample Size)

Taconite “B” and SafeLane were the best performers losing about half as much mass compared to the other three samples. Taconite “A” lost the largest amount of mass during testing.

Freeze thaw damage indirectly measures the porosity of aggregate particles. Pore space in an aggregate particle allows for water penetration. The presence of water in pore space during freezing temperatures leads to aggregate degradation. Damage to an aggregate particle is caused from the abrasion of ice while water molecules are expanding from freezing.

**Sieve Analysis**

A gradation for each of the five samples was conducted and compared to the MnDOT chip seal requirements shown in Table 14.

.

Table 14: MnDOT Chip Seal Gradation Requirements

| Sieve Size | % Passing by Weight |
|---|---|
| No. 4 | 100 |
| No. 8 | 30 – 75 |
| No. 16 | 0 – 5 |
| No. 30 | 0 – 1 |

In Figure 22 it is shown that the Poly-Carb systems all have similar gradations. The main difference in the systems the amount retained on the #8 and #16 sieves. SafeLane's gradation is significantly different than the Poly-Carb system's gradation. It is also the only aggregate sample to follow the MnDOT chip seal requirements.

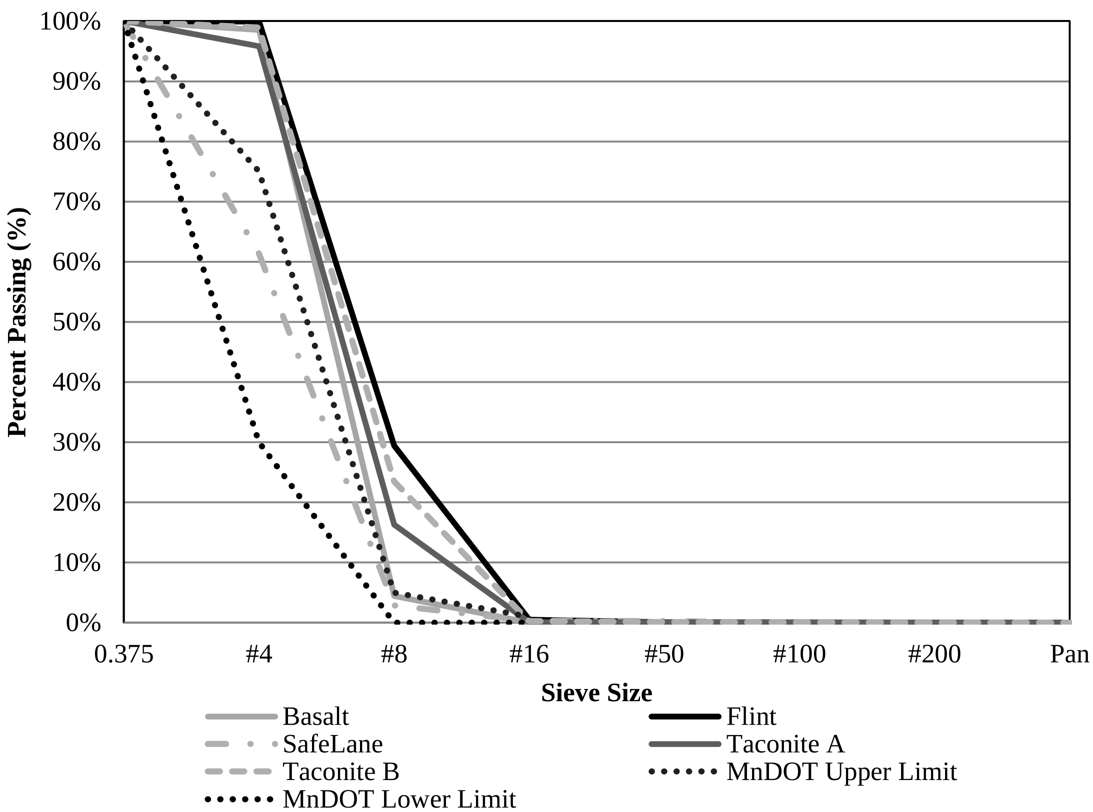


Figure 22: Sieve Analysis Compared to MnDOT Chip Seal Requirements

The most important aspect of the sieve analysis is that all aggregate samples have a majority of their particles falling bewteen the 9.5 mm and #16 sieve. This is created to allow for an open gradation that provides a high macrotexture when placed in the HFO binder.

## Aggregate Laboratory Testing Summary

A number of laboratory tests were conducted to gain perspective on aggregate samples used in conjunction with epoxy and asphalt binders for high friction overlay systems on bridge decks. The main concerns regarding aggregates assessed through this testing are the strength to endure abrasion, and to provide both a solid macro- and microtexture during the service life of HFO systems.

All five aggregate samples performed satisfactorily through the battery of tests conducted in this task. While all five aggregates showed good physical and mechanical properties on a relative basis, the best abrasion performance was observed in basalt, flint, and taconite “A”. The aggregates with the best freeze thaw performance were taconite “B”, and SafeLane. Overall all aggregate is expected to perform well in HFO systems.

# Chapter 4: Field Observations and Testing

This task includes field observations and testing conducted on the different HFO systems. Systems looked at in this research were designed to provide a strong, waterproof, and skid resistant traffic surfaces. The main areas of concern in context of the performance of wear courses include: (1) retention of aggregate particles, (2) degradation and polishing of aggregate particles due to the actions of traffic loading and snow plow abrasion, and (3) durability of aggregate under repeated freezing and thawing conditions. Field tests were conducted to ascertain the effects of the aforementioned areas of concern. The scope of testing is contained in the following section. Test procedures, observations, field test results, and comparisons are in subsequent sections and finally, a summary is presented. This task is similar to several other studies that have been conducted regarding the use and performance of high friction overlay systems (Anderson et al., 2012).

## Scope of Observations and Field Testing

The scope of this project is to provide a comparative analysis of different systems, conditions, and performances. This comparison is based on two years of field testing and observations conducted at bridge sites where HFO systems have been exposed to the climatic conditions and roadway maintenance operations prevalent in Minnesota and similar cold climate regions. Data from Year-1 and Year-2 will be presented. The results will not only compare the different systems and bridges against one another, it will also compare the systems against the proclaimed benefits of an HFO system. The dates for the site visits are listed below in Table 15.

Table 15: Site Visit Dates

| Bridge Number | Date Visited | |
|---|---|---|
| | Year-1 | Year-2 |
| 6347 | N/A | 10/3/14 |
| 9066 | 9/16/13 | 6/2/14 |
| 9067 | 9/16/13 | 6/2/14 |
| 9823 | 8/29/13 | 8/6/14 |
| 14809 | N/A | 6/4/14 |
| 14810 | 9/17/13 | 6/4/14 |
| 27019 | 9/13/12 | 8/20/14 |
| 27758 | N/A | 10/2/14 |
| 69006 | 8/28/13 | 6/18/14 |
| 86013 | 9/13/12 | 8/20/14 |
| 86019 | 9/13/12 | 8/20/14 |
| 4190 | 9/4/13 | N/A |
| 5718 | 10/1/13 | 5/28/14 |
| 9036 | 9/4/13 | N/A |

## Observations

The observation of an HFO system in service is equally as important as the standard tests completed. The observations compliment test results and also help to explain why specific test results or trends have occurred. The following section provides a discussion of such observations including distresses that have occurred and differences between HFO systems.

### Distresses

Distresses in the HFO systems are one of the most common parameters observed during site visits. The inspection of different distresses is important since distresses may cause issues that reduce the life of HFO. The following subsections describe each type of distress observed with examples from different bridges.

***Construction Joints and Uneven Aggregate Distribution***

Uneven distribution of aggregates was apparent in several of the systems visited. This is caused by the manual process of adding aggregate to the epoxy matrix. It is difficult to evenly spread aggregate with shovels. Applying too much aggregate in an area leads to insufficient epoxy coating on the aggregates, and without proper bonding, these aggregates are more prone to popping out of the matrix. An insufficient amount of aggregates in the matrix creates areas prone to polishing. The construction joint in Figure 23 is the line running down the center of the figure that looks similar to a crack. This joint was caused by either a pause in construction or the application of too much aggregate in one spot. Construction joints are mostly a cosmetic distress unless an insufficient amount of aggregates are placed which can affect the skid properties.

Figure 23: Construction Joint (Bridge 9066)

This uneven application can lead to higher amounts of distresses during the life span of an HFO system. Bridge 14809 provided a good example of how the quality of construction during installation can play a role in the performance of epoxy overlay systems. Such a difference is depicted below in Figure 24. The view of the picture is looking against the direction of traffic. A construction joint exists in the middle of the picture. The system on the left side of the picture is a bit lighter in color. On the right side there appears to be a higher rate of raveling. This is shown best when looking at the bottom right side of the picture.


Figure 24: Differences on Both Sides of the Construction Joint (Bridge 14809)

***Excessive Placement of Epoxy***

Another type of distress caused by construction is polished areas due to the excessive placement of epoxy. This distress occurred on Bridge 6347 where a significant amount of polished areas existed. Looking along the wheel path in Figure 25, the darker areas are where polished areas exist. The magnified section of this figure provides an example of the difference in texture between a polished (left side) and normal (right side) area. As vehicle traffic and snow plows cause abrasion to the roadway, the aggregates are broken along the plane of the epoxy leaving the polished area.

Figure 25: Polished Areas along Bridge 6347

This abrasion along the epoxy plane is exemplified in Figure 26 where the right half of the picture includes an area where the epoxy level is near the top of the aggregate layer. Any aggregate exposed in this heavy epoxy area is now flush with the top of the epoxy due to abrasion.

Figure 26: Proper Epoxy Levels (Left Side) Next to Area with Excessive Epoxy (Right Side)

### *Snowplow abrasion*

A common distress in snowy regions, such as Minnesota, is damage caused from the service of snowplows. Driving a snowplow blade across a roadway can cause severe abrasion and involves a series of impact events. Damage most often occurs at the crown of a roadway or in between rutted wheel paths. This snowplow abrasion is shown in Figure 27. The damage is most prevalent along the centerline as well the right wheel path in the driving lane. On this bridge the abrasion removes some of the aggregate which gives the roadway the appearance of a lighter color. Snow plow abrasion is detrimental to HFO systems because even with the use of aggregates that are proficient against deterioration due to abrasion, the bond between epoxy and aggregates are not strong enough to resist the shear force a plow blade imposes. Also if aggregate is being removed from the surface, the potential safety benefit quickly diminishes.

Figure 27: Transverse Snowplow Abrasion (Br 4190)

The macrotexture of Bridge 27758 still exists along parts of the bridge, but heavy abrasion has taken its toll on the overlay system. As shown below (Figure 28), areas of the bridge have minimal aggregate pop-out but finer material has been removed between the “larger” aggregate exposing the bridge deck. Some locations have the opposite distress where severe aggregate pop-out exists and leaving polished areas as well as areas with minimal aggregate particles (Figure 29). Other areas of this bridge have parts of the overlay system that are completely missing.

Figure 28: Existing Macrotexture on Bridge 27758


Figure 29: Polished Area on Bridge 27758 along with Transverse Cracks

One distress on Bridge 27758 that has not occurred with any other system is the complete removal of epoxy and aggregates from the bridge deck surface due to abrasion. The removal from snow plow abrasion has occurred near joints or at the edges of other systems but not in the middle of the system as seen on Bridge 27758. Depicted below in Figure 30, the localized abrasion has removed strips of the system. The pattern of removal suggests that the epoxy was removed due to the high impact of a bouncing snow plow blade. This system is thinner than any other system in the study, with a thinner cross section less removal of material is required to reach the deck surface.

Figure 30: Excessive Snow Plow Abrasion (Br 27758)

The thin cross section of this system is shown in Figure 31. In this picture there is not a distinguishable elevation difference between the bridge deck and the epoxy overlay.

Figure 31: Thin Epoxy System (Br 27758)

In the Novachip systems, the worst snowplow abrasion came from an uneven roadway elevation. The right side of the Novachip system on Bridge 86019 (Figure 32) has a significant amount of abrasion compared to the left side. During the site visit, settlement was visible on the left side of the bridge suggested by uneven approach panels and mismatched Jersey barrier heights. This settlement has lowered the left side of the bridge causing uneven seating of snowplow blades on the right side. This seating increases the rate of abrasion compared to the left side of the system.

Figure 32: Uneven Snowplow Abrasion due to Uneven Roadway Elevation (Br 86019)

The area most damaged on this system is along the expansion joints. A common distress among the different epoxy systems, snowplows seem to cause catastrophic deterioration near joints. As shown in Figure 33, traffic is flowing from left to right. The right side of the figure is the concrete departure panel of the bridge. More area is missing on the departure side of the expansion joint.

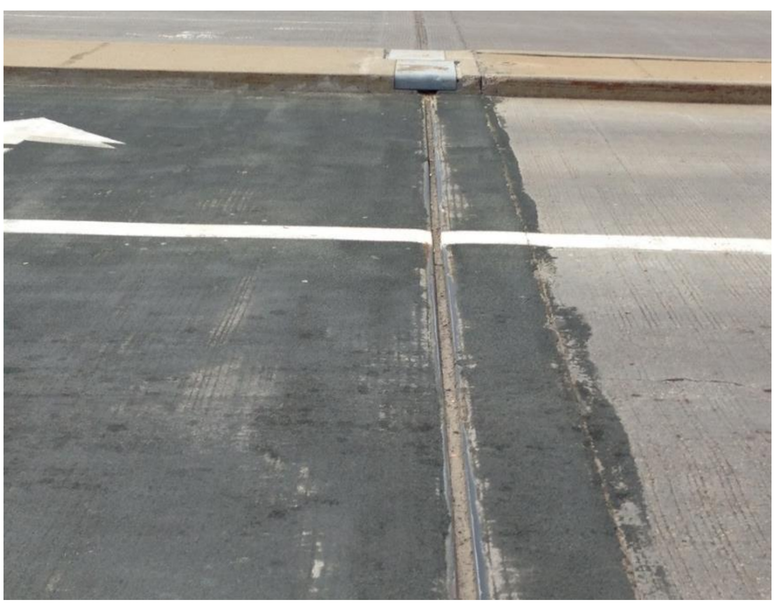

Figure 33: Missing Overlay System near Expansion Joints on Transpo System (Br 27758)

Damage at the joints is common on all HFO systems. The systems are not constructed to be flush with joints, so abrasion due to traffic and snowplows causes a raveling distress by removing aggregate from the epoxy. This damage is apparent in Figure 34. The figure shows a joint from Bridge 69006. In the figure, traffic is moving from left to right. As the snow plow blade crosses the joint it drops off the epoxy system until it is flush with the joint. It then scrapes away the system on the other side of the joint.

This damage is also shown in Figure 35 with a joint from Bridge 9066. Bridge 69006 and 9066 had their overlay systems placed in 2009 and 2010 respectively. So the damage in Figure 34 is from three winters and the damage in Figure 35 was caused over two winters.

Figure 34: Joint Abrasion on Bridge 69006 (Traffic Moves from Left to Right)

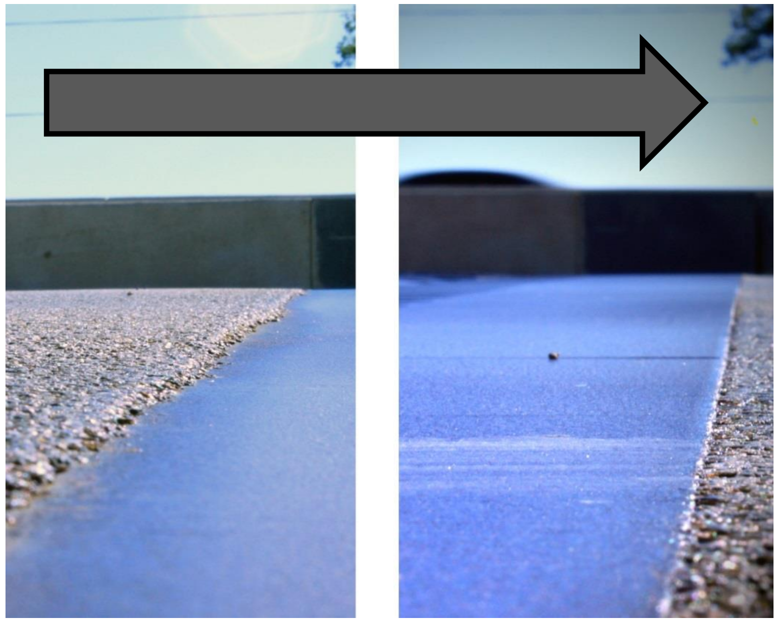

Figure 35: Joint Damage on Bridge 9066 (Traffic Moves from Left to Right)

Damage is also produced as snow plows cross from approach panels to the bridge deck. The mismatch between the approach panel and the bridge deck causes part of the epoxy system to be scrapped off the bridge deck (Figure 36). Transverse abrasion from snow plows in high elevation areas is also apparent in Figure 36.

Figure 36: Damage near Approach Panels on Poly-Carb System (Bridge 69006)

As shown in Figure 37, the HFO system removal due to snow plows occurs across all HFO types. The removal in Figure 37 is especially bad due to the uneven elevation between the roadway and the approach panel. The asphalt roadway on the left side of the figure has settled exposing the concrete approach panel and overlying HFO system.

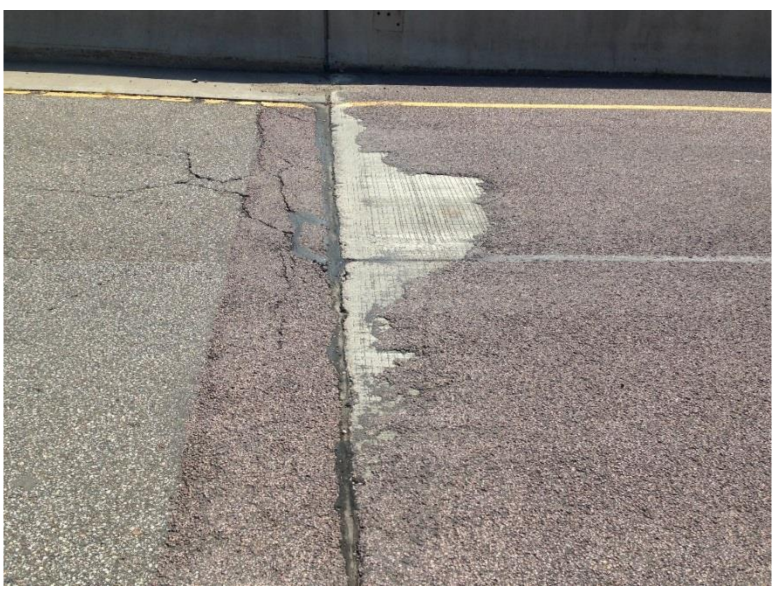

Figure 37: System Removal near Joint on NovaChip System (Bridge 86019)

Due to the thin nature of the epoxy systems, aggregates that are placed on the top layer may not receive adequate epoxy cover to stay embedded in the system. Abrasion from snow plows and traffic will remove aggregate from the epoxy matrix. This distress was common across all epoxy bridges.

### *Aggregate Raveling*

Removal of aggregates from the HFO system is due to inadequate binding to the system. Abrasion forces are most commonly responsible for aggregate removal in the epoxy based systems: Poly-Carb, SafeLane, and Transpo. Aggregate raveling was more apparent in the wheel path and was more common in polished areas. These systems had large amounts of aggregate raveling at every bridge location. Figure 38 contrasts a surface with the proper aggregate coating against a surface with significant amount of polishing and raveling.

Figure 38: Texture Comparison of Br 4190 (Left: Proper Aggregate Macrotexture, Right: Aggregate Raveling)

In Figure 39 there is a comparison of a typical section along the driving lane for three different systems. On the left is Novachip (Br 86013) which has minimal aggregate raveling compared to its epoxy counterparts. In the middle is a typical SafeLane surface (Br 14810), and a typical Poly-Carb surface (Br 9067) on the right. The reason Novachip systems have minimal loss of their aggregates is due to the fact that the system is a thin-bonded open-graded asphalt overlay. The asphalt binder has viscoelastic properties whereas the epoxy based systems are much stiffer elastic-plastic response. The viscoelastic properties may allow the asphalt system to absorb the shock exerted by snow plow action.

Two different scenarios occur when a shearing force such as a snow plow pushes across the top of the overlay surface. The first occurs with asphalt system. When forces are applied an aggregate is allowed to compress into the asphalt binder at a microscopic level. The asphalt then has an elastic recovery after the force is removed placing the aggregate back into its original location. This elasticity of asphalt allows the system to absorb some of the energy caused by abrasion. In the second scenario with elasto-plastic epoxy, the system cannot compress and cannot absorb the energy as the stiffness of epoxy is very high. This results in the applied forces breaking the bonds formed between the epoxy and aggregates and removing aggregates from the system.

Novachip systems are constructed similar to asphalt overlays, thus the system is compacted with a roller during construction. This compaction helps to push the aggregates further into the asphalt matrix and achieve a higher bond. Additionally, asphalt remains pliable while it ages, therefore aggregate in the Novachip systems are able to compress into the asphalt under a load even after the initial placement. The continued embedment of aggregate into the asphalt membrane is noticeable in the Novachip systems (Figure 39, left). These two features prevent the excessive raveling found in epoxy systems. In an epoxy system, once hardened, the aggregates do not have the capability to compress further into the epoxy matrix, which may lead to protruding aggregate. These protruding aggregates are more susceptible to abrasion, polishing, and raveling.

Figure 39: Comparison of Loss of Aggregate between Novachip (Left), SafeLane (Middle), and Poly-Carb (Right) Systems

### *Aggregate Polishing*

Aggregate polishing was a common distress found in the wheel paths. An extreme case of polishing was found on Bridge 9066. In Figure 40 and Figure 41 the reflection of sunlight on polished areas makes it much easier to spot locations with poor friction.

Figure 40: Wheel Path Polishing (Br 9066)

Figure 41: Close View of Polished Area (Br 9066)

***Cracking***

Similar to concrete and asphalt pavements, cracks in HFO systems also occur. Several hairline cracks exist in the Poly-Carb and SafeLane systems but none of the cracks appeared to propagate through the entire thickness of the system. In the Transpo systems, cracks that propagated through the entire system did occur. On Bridge 27758 (Penn Ave) the cracks appear to be due to the system being thinner than any other system allowing reflective cracks from the underlying bridge deck to propagate through the system. These cracks are shown below in Figure 42.

Figure 42: Transverse Cracks on Bridge 27758

Cracking also occurred on Bridge 6347. This cracking was completely through the thickness of the HFO systems. The approach panels on either side of the bridge have a transverse crack near the expansion joints. The transverse crack occurs along the back edge of the concrete abutment wall (Figure 43) which suggests that settlement behind the abutment has occurred. The settlement lowered and cracked the approach panel and the HFO system. This settlement may also have detrimental effects on the expansion joint and could render the joint ineffective.

Figure 43: Transverse Crack near Expansion Joint of Bridge 6347

Transverse cracking (Figure 44) and reflective cracking along expansion joints (Figure 45) both existed in the Novachip systems. The Novachip systems are asphalt overlays and thus distress differently than the other epoxy systems. Novachip systems distress similar to conventional asphalt pavements. The cracks reflected through the Novachip systems from existing cracks or settlement in the approach panels and bridge decks. The placement of Novachip systems on well-performing bridge decks would eradicate a majority of the crack distresses observed in this study.

Figure 44: Longitudinal Crack in Novachip System (Br 27019)

Figure 45: Reflective Cracking in Novachip System (Br 27019)

Several different types of cracking existed. Cracking that propagates through the entire HFO system is detrimental because it provides an area where water and chloride ions are able to penetrate into the concrete deck.

**Delamination of High Friction Overlay Systems**

Several bridges in this study have sections where the epoxy overlay has delaminated from the bridge deck. On Bridge 6347 the delamination of the epoxy overlay system has occurred along the centerline of the roadway. The majority of the delamination has occurred along the construction joint which runs down the middle of the bridge. Some of the worst delamination is shown below in Figure 46. The construction joint can be seen in between the no passing lines.

Figure 46: Delamination of Transpo System on Bridge 6347 along Centerline

Delamination also occurred in several Poly-Carb systems (Bridge 4190 and 9036). Depictions of such delamination are shown in Figure 47 through Figure 49.

Figure 47: Delamination of HFO on Robert Street Bridge (Br 9036)

Figure 48: Minimal Bond Strength Existed in the Distressed Areas (Br 9036)

Figure 49: Delaminated Area during Maintenance (Br 4190)

During the first year of investigation, Bridge 4190 (TH 55 in St. Paul, Poly-Carb) and Bridge 9036 (Robert St in St. Paul, Poly-Carb) were visited in addition to the bridges in this study. Both bridges had delamination of their HFO systems. After visiting these two bridges they were repaired however delamination occurred again on these epoxy systems during the second year of this study (2014). The two bridges were not visited during the second year of this study (2014).

There appears to be two similarities between the bridges where this delamination occurs, crossing water and lower functioning expansion joints. The epoxy systems crossing water in this study are found on Bridges 6347 (TH 243 in Otsego, Transpo), 9066, and 9067 (I-94 in Moorhead, Poly-Carb). Delamination has occurred on Bridge 6347 but not on either 9066 or 9067. One main difference between Bridges 9066, 9067 and the other bridges with delamination is the expansion joints. On Bridge 9066 and 9067 the expansion joints are significantly larger compared to the other bridges. The expansion joint of Bridge 9066 is shown below in Figure 50.

Figure 50: Expansion Joint on Bridge 9066

The expansion joints of bridges with delamination are smaller in comparison, and in some cases do not appear to be in the best condition. The Robert Street Bridge expansion joints (Figure 51) did not appear to be functioning well. The smaller joint, shown on the top of Figure 51, was south of the delamination and appeared to be sealed. The northern joint (Bottom of Figure 51) was larger but contained a significant amount of fine incompressible material in the joint. A similar joint, in similar condition, is found on Bridge 4190 (TH 55) and is shown in Figure 52. This joint was also filled with fine material.

The constant presence of water under a bridge with an epoxy system may play a role in the delamination of the system. Moisture from the river may infiltrate from the underside of the bridge and reduce the bonding between the epoxy overlay and the bridge deck. As previously discussed in the Literature Review, moisture may be present in the concrete deck even if water is not visually apparent on the surface.

Concrete and epoxy have different coefficients of thermal expansion thus expanding at different rates. This difference causes internal pressures on the epoxy system during thermal expansion and the pressure is further compounded with the presence of small or ill-functioning expansion joints. If moisture is present in the bridge deck during installation, a weaker bond between the epoxy and concrete may occur. Internal stresses, emphasized by small or ill performing joints, may be enough to delaminate HFO systems with a weaker bond.

Bridge 27019 and 86019 (Otsego, Novachip) also cross water however the Novachip systems, with their elastic properties from asphalt, would be able to absorb the internal stresses compared to the rigid epoxy system. As discussed in the previous section, Bridge 6347 (TH 243 in Osceola, Transpo) has settlement behind its abutment which could be interfering with its expansion joints ability to operate.

Figure 51: Robert Street (Br 9036) Expansion Joints, Both North (Bottom) and South (Top) of Delamination

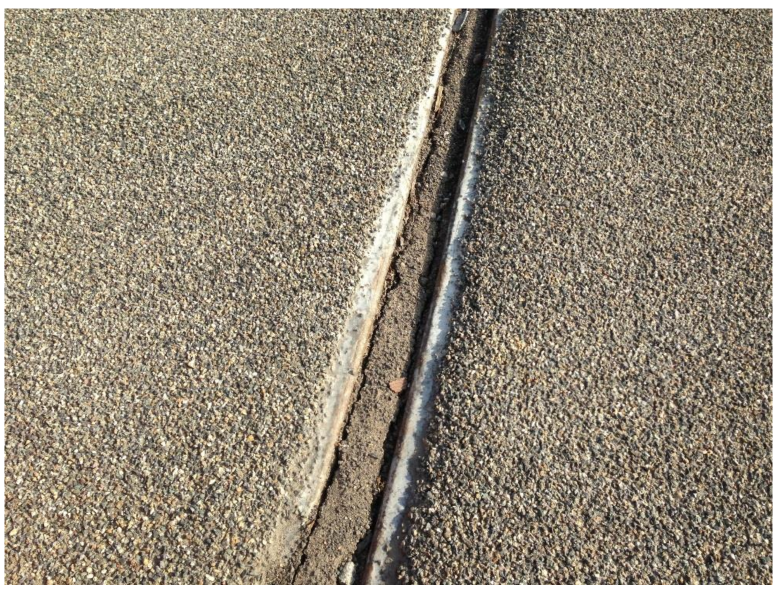

Figure 52: Bridge 4190 (TH 55) Expansion Joint

When cracks or delamination occur in HFO systems their ability to protect the underlying deck from chloride infiltration is compromised. This infiltration can lead to reinforcement steel corrosion.

### Differences in Thickness of High Friction Overlay Systems

In this study, the different HFO systems have different thicknesses. The thickest system is Novachip (3/4 inch [19mm]), followed by SafeLane (3/8 inch [9.5mm]), and then Poly-Carb (1/4 inch [6.5mm]). A visual of the difference in thickness is provided below (Figure 53). The figure shows steel plate samples that removed a section of an HFO system during the torque bond testing. A Poly-Carb system sample (Left) is side by side with a Novachip system sample (Right). Since epoxy has a higher tensile strength compared to asphalt, epoxy systems can have smaller cross-sections and still achieve adequate binding to the bridge deck as well as the aggregates in the HFO system. The slight difference in thickness between SafeLane and Poly-Carb systems is due to the selected aggregate size. SafeLane is slightly thicker because it uses larger aggregates in its system compared to Poly-Carb.

Figure 53: System Thickness Comparison. Left: Poly-Carb System, Right: Novachip System

The main advantage for thinner epoxy systems is a reduction in dead load on bridges. Assuming the HFO systems have a relatively similar density, epoxy systems place half the weight on a bridge deck compared to asphalt based systems.

## Testing Procedure and Methodology

Testing was completed on overlay systems in order to acquire friction, permeability, and bond strength properties. The tests and their corresponding ASTM designations or procedures are described in Table 16. Several testing locations along the length of the bridges were used to collect data and reduce bias. At each test location the bridge decks were tested in both wheel path and non-wheel path areas. In general the wheel path was located by measuring 2.5 feet [76 cm] into the lane from the fog line. The non-wheel path's location was considered to be 6 feet [1.8 m] from the fog line. In some instances wearing of the deck due to wheel loads were used to identify the wheel path.

Table 16: Properties and Test Method

| Property | Test Method |
|---|---|
| Pavement Macrotexture Depth | ASTM E 965-96 Standard Test Method for Measuring Pavement Macrotexture Depth Using a Volumetric Technique |
| Torque Bond Strength | There is currently not an ASTM standard for this test. However results are to be used for comparison of systems within this project only. |
| Pavement Permeability | Testing was performed based on the operating manual for the NCAT Asphalt Field Permeameter Kit AP-1B which was used for testing |
| Skid Resistance of Paved Surface | ASTM E 274 Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire |
| | ASTM E1911-09 Standard Test Method for Measuring Paved Surface Frictional Properties Using the Dynamic Friction Tester |
| Chloride Ion Penetration | AASHTO T277 Standard Method of Test for Rapid Determination of the Chloride Permeability of Concrete |

**Pavement Macrotexture Depth**

The ASTM 965-96 test method finds the mean texture depth of a pavement through a volumetric procedure. A known volume of evenly graded silica sand is spread evenly over an area of pavement, with the known sand volume and surface area of the space it fills one can solve for the mean texture depth (MTD). This MTD is used to compare the texture of HFO systems. A larger MTD means more texture and inherently higher friction. An example of this process is shown below in Figure 54. The equation used to calculate the mean texture depth and other data used is provided in the Appendix.

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Figure 54: Mean Texture Depth

**Torque Bond Strength**

In this test a steel plate is bonded to the overlay system using a fast setting epoxy. Once this epoxy is set (approximately two hours) the plate is removed using a torque wrench. This wrench has a readout that provides the torque required to remove the plate. This test measures how well a system is bonding with the concrete bridge deck. Removal of the epoxy system when torque is applied advocates for a poor bond between the system and bridge deck. Figure 55 shows the torque wrench attached to a plate just before testing. Figure 56 and Figure 57 show plates with debonded overlay material at Bridges 9036 and 14810 respectively.

Figure 55: Torque Wrench and Attached Plates (Br 69006)

Figure 56: Plate with Debonded Overlay (Br 9036)

Figure 57: Plate with Debonded Overlay (Br 14810)

During the first year of bridge deck wear course evaluation, the torque bond strength testing was conducted using a two inch [5 cm] extension piece to connect the end of the wrench with the steel plate. During the second year of testing this extension was removed and the wrench was directly connected to the steel plate. This brought the wrench closer to the steel plate. Since this distance was reduced, the force applied to the wrench was more effectively transferred into the torque placed on the steel plate. With the extension piece in place it is believed that some force applied to the wrench was exerted in a bending mode rather than pure rotational torque when transferring force from the wrench to the steel plate, likely resulting in a reduction of torque required during testing. For this reason the torque values from Year-1 should not be directly compared to the values found in Year-2.

**Pavement Permeability**

This test is performed to provide the coefficient of permeability of a pavement. This is accomplished by timing the rate at which the water level decreases in the testing equipment shown below in Figure 58. The purpose of the epoxy system is to provide a waterproof system to add protection to the bridge deck. Some deicing chemicals used on roadways have detrimental effects on the concrete and reinforcement bars in the bridge deck. By providing an impervious surface, these soluble chemicals are unable to infiltrate the bridge deck and thus reduce the degradation of the infrastructure. It is important to visually inspect the epoxy system and perform permeability tests on any surface cracks in the system to find if water is able to infiltrate the epoxy system. Figure 58 is a test performed on a crack in the system. However as shown in the bottom of the figure, the crack does not extend through the entire thickness of the system. Evidence of this is the water permeating back to the surface outside of the testing equipment. The equation used to calculate the coefficient of permeability and other data used is provided in the Appendix.

Figure 58: Permeability Test over Cracked Overlay (Br 9823)

## Skid Resistance of Paved Surfaces

Two different tests were completed to find the skid resistance of the HFO systems. The first test found resistance values using a skid trailer. This test was conducted by MnDOT and was completed during both Year-1 and Year-2.

The second test measured resistance values using the Dynamic Friction Tester. Testing was only completed during Year-2 of the study and was conducted by MnDOT's Office of Materials and Road Research. Below are descriptions of the two different testing procedures.

### ***Measuring Surface Frictional Properties with a Skid Trailer***

An apparatus with a test wheel and recording instrumentation is driven over a roadway at a determined speed. While at this speed, the test wheel has its brakes locked. The force required to slide the locked test tire over the roadway is used to determine the skid number. Water is sprayed onto the roadway in front of the test tire in order to simulate the "worst case" scenario of a wet road. Minnesota Department of Transportation (MnDOT) collects skid numbers for roadways using a KJ Law (Dynatest) Skid Trailer. Two types of tires are used during testing, ribbed tires; which have a tread design, and smooth tires. The ribbed tire is more sensitive to microtexture while the smooth tire is more sensitive to macrotexture (M. Watson, 2011). Smooth tires also display more variation to water film depth (FHWA, 2010). Although this test is conducted to find the surface friction, a certain skid number does not necessary correlate to a safe or unsafe road condition (FHWA, 2010). A major drawback of the skid trailer is its use on a tangent curve. When driving on a tangent curve the weight of the skid trailer is not perpendicular to the road surface due to the super elevation of the roadway, thus skewing the coefficient of friction results. It may also be unsafe to test some tangent curves because traffic simply cannot drive at the test speed required on some curves.

### *Measuring Surface Frictional Properties with the Dynamic Friction Tester*

This test is performed using an apparatus that houses a rotating disk with three rubber sliders attached on the bottom of the disk. The test procedure is standardized through ASTM E1911-09 specification. The disk is mounted in a plane parallel to the driving surface. The disk is rotated and brought up to its maximum tangential velocity of 90 km/h (55 mph). Once the disk is up to its maximum tangential velocity, water is pumped in front of the rubber sliders as the disk is lowered to make contact with the surface. The torque is monitored as the friction between the roadway and rubber sliders reduce the disk's rotational velocity. The friction at various tangential velocities is recorded for later analysis. The testing apparatus is depicted below in Figure 59. The piece of equipment placed on the roadway surface placed to the right of the bucket in Figure 59 houses the rotational disk. Each test location ran five repetitions of the Dynamic Friction Tester (DFT) test. Tests were run at one to three different locations for both the wheel and non-wheel path regions of each bridge. The number of test-runs was based on time constraints due to traffic control.

Figure 59: Dynamic Friction Tester

There are some benefits for the use of the DFT as compared to a skid trailer. The first is reliability and ability to quantify repeatability. The DFT is set up and used in a singular location. This allows for multiple tests to be run in the same location for higher precision.

It also eliminates error that may interfere with test results. This error can be caused by the bouncing or lateral movement of the skid trailer which must be in motion for its testing procedure. The fact that testing is done in a static location for DFT testing provides the opportunity to test more accurately in wheel and non-wheel paths. Draw backs for the localization of this test include that the friction values of a surface are based on a very small percentage of the surface, however for this study the higher accuracy allowing us to compare multiple bridges and systems outweighs the disadvantage.

**Chloride Ion Penetration**

This test is conducted to test the content of chloride ions present in bridge decks with an HFO system. The purpose of this test is to determine if HFO systems protect bridge decks from chloride ion penetration which facilitates corrosion of reinforcing steel inside the concrete deck. Samples are cored out of the bridge deck and vary from four to six inches in depth. Each sample is cut at corresponding depths, crushed, and tested for the chloride ion content according to the AASHTO T-277 test method. These cuts are made to determine the chloride levels present at different depths throughout the bridge deck. An illustration of where a core sample is cut is shown below (Figure 60).

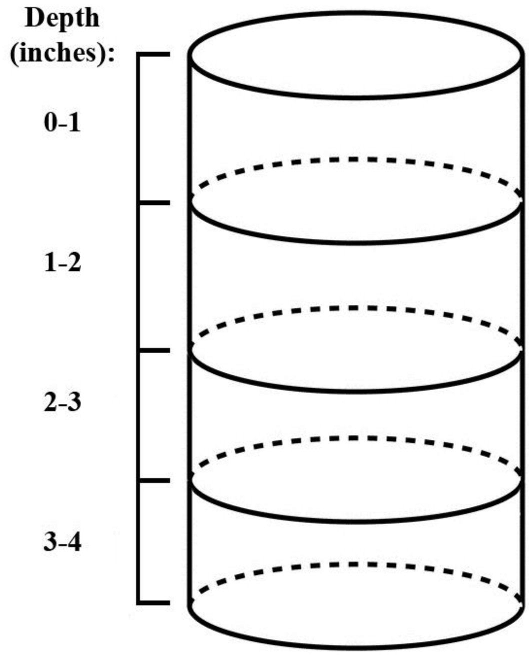


Figure 60: Cutting Configuration of a Sample Core

The typical range at which corrosion may begin is 0.59 to 0.70 kg/m$^3$, or 250 to 330 parts per million (Costa, 2010). The chloride content threshold of 0.6 kg/m$^3$ was identified by a Federal Highway of Administration (FHWA) corrosion study as the lower value where corrosion can occur (Clemena, 1998). The important depth to look at is the depth closest to reinforcing steel (Young, Durham, & Bindel, 2012). For concrete bridge decks this depth is two inches, the minimum cover required for reinforcing steel (AASHTO, 2012). The purpose of this testing is to see whether HFO systems are protecting the bridge decks from chloride ion penetration.

For this testing, sample cores were removed from the field by MnDOT. The preparation and testing of samples was conducted by the MnDOT Office of Materials and Road Research. A limited amount of samples were removed for this testing. Core samples were collected from Bridge 9823 (Atkinson, MN, Poly-Carb system), and Bridge 69006 (Virginia, MN, Poly-Carb system). Tabulated below in Table 17 are the year and corresponding number of sample cores taken from the bridge decks. Samples are cored out of the bridge deck and vary from four to sixes inches in depth.

Table 17: Number of Sample Cores Taken in Various Years

| Bridge | Year | Number of Sample Cores | |
|---|---|---|---|
| | | Driving Lane | Passing Lane |
| 9823 | 2011 | 3 | 3 |
| | 2014 | 1 | 1 |
| 69006 | 2010 | 3 | 3 |
| | 2011 | 3 | 3 |
| | 2014 | 1 | 1 |

The 2010 and 2011 cores were cut at every inch along the deck's depth. The 2014 cores were cut at every half inch for the first two inches, and every inch after. The results from this testing are discussed in the following section.

## Results

The field results and their analysis for the different bridges and their HFO systems are below. Additional equations, data, and calculations are presented in the Appendix.

### Pavement Macrotexture Depth Results

The following section discusses the results found from the pavement macrotexture depth testing. This test is useful for comparing different physical characteristics of the bridges and how they affect the mean texture depth (MTD). Three lines are placed on Figure 61 and Figure 62 as a baseline comparison against normal bridge surfaces. Table 18 summarizes the attributes of these baseline surfaces.

Table 18: Baseline Surface Texture Attributes

| Surface Type | MTD (in) | Remarks |
|---|---|---|
| New Asphalt | 0.0161 | Taken within 6 months of construction |
| New Brushed Concrete | 0.0295 | Taken within 6 months of construction |
| Worn Tined Concrete | 0.0138 | Low speed bridge deck surface after 14 years of service with an ADT of less than 200. |

The mean texture depths are found below for both the driving lane (Year-1: Figure 61, Year-2: Figure 62) and passing lane (Year-2: Figure 63). There are two general trends for the mean texture depth. These trends are divided by the different types of HFO systems. The first trend is for the epoxy based systems Poly-Carb (Bridges 9066, 9067, 9823, and 69006), Transpo (Bridges 6347 and 27758), and SafeLane (Bridges 14809 and 14810). In this trend the mean texture depth varies with the smallest depth in the wheel path, the largest depth in the shoulder. The non-wheel path has a depth somewhere in between. The logic behind this trend is that snow plow abrasion causes more severe abrasion on driving lanes lowering its depth compared to the shoulder. The wheels from traffic further abrade the wheel path lowering its depth compared to the non-wheel path. This trend exists in both the driving and passing lanes.

The second trend is for the Novachip system (Bridge 27019 and 86013). On these systems the shoulder depth is smaller compared to driving lanes. The reason for this is the maintenance operations performed on these bridges. Snow is plowed onto the shoulders and then removed over the edge using a snow thrower. The blades on the snow thrower cause additional heavy abrasion on the shoulder.

On Bridges 9036 (Poly-Carb), 27758 (Transpo) and 86013 (Novachip) the wheel and non-wheel path depths are very comparable. A small variance between the wheel and non-wheel path is reasonable since this is a lower traffic volume, lower speed bridge. On Bridge 27019, the wheel path has a larger depth compared to the non-wheel path. A possible explanation for this is the existence of a construction joint along the non-wheel path section tested.

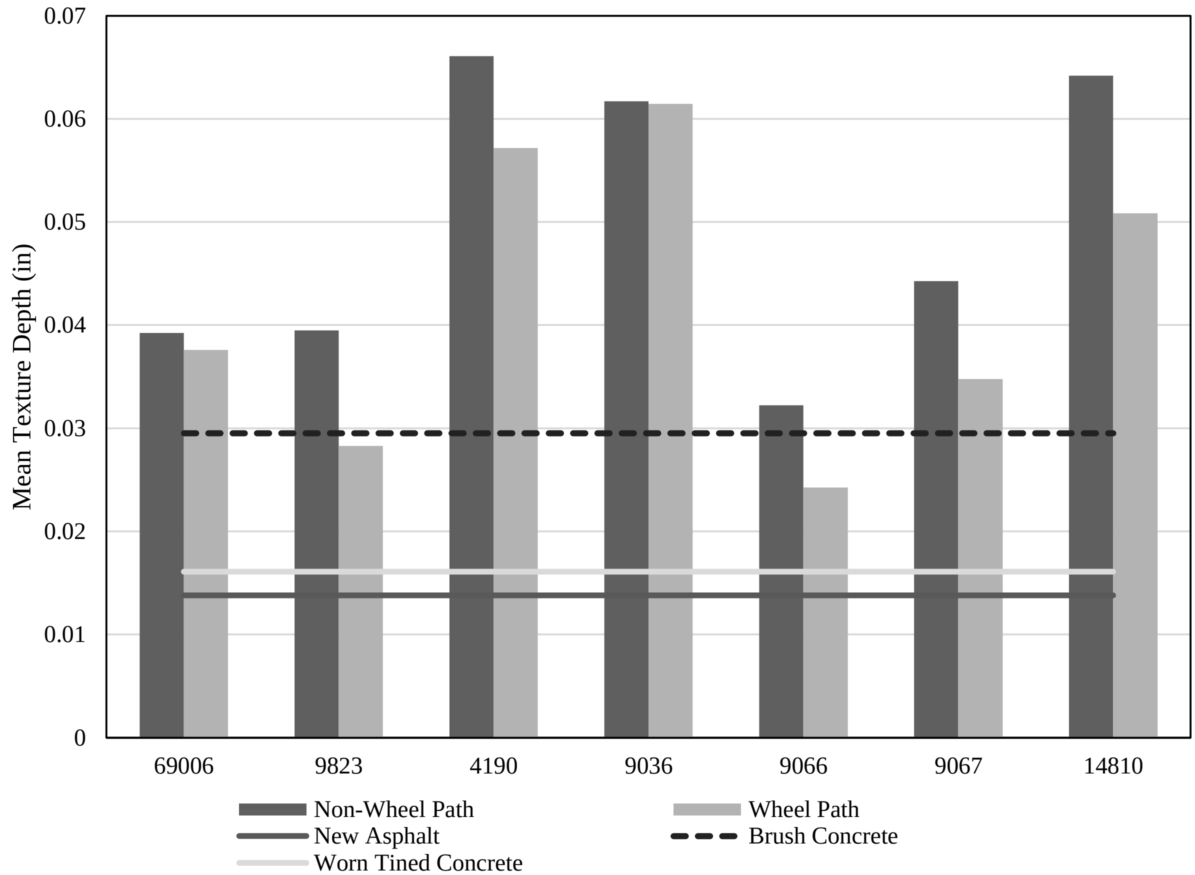


Figure 61: Mean Texture Depth (Year-1: Driving Lane)

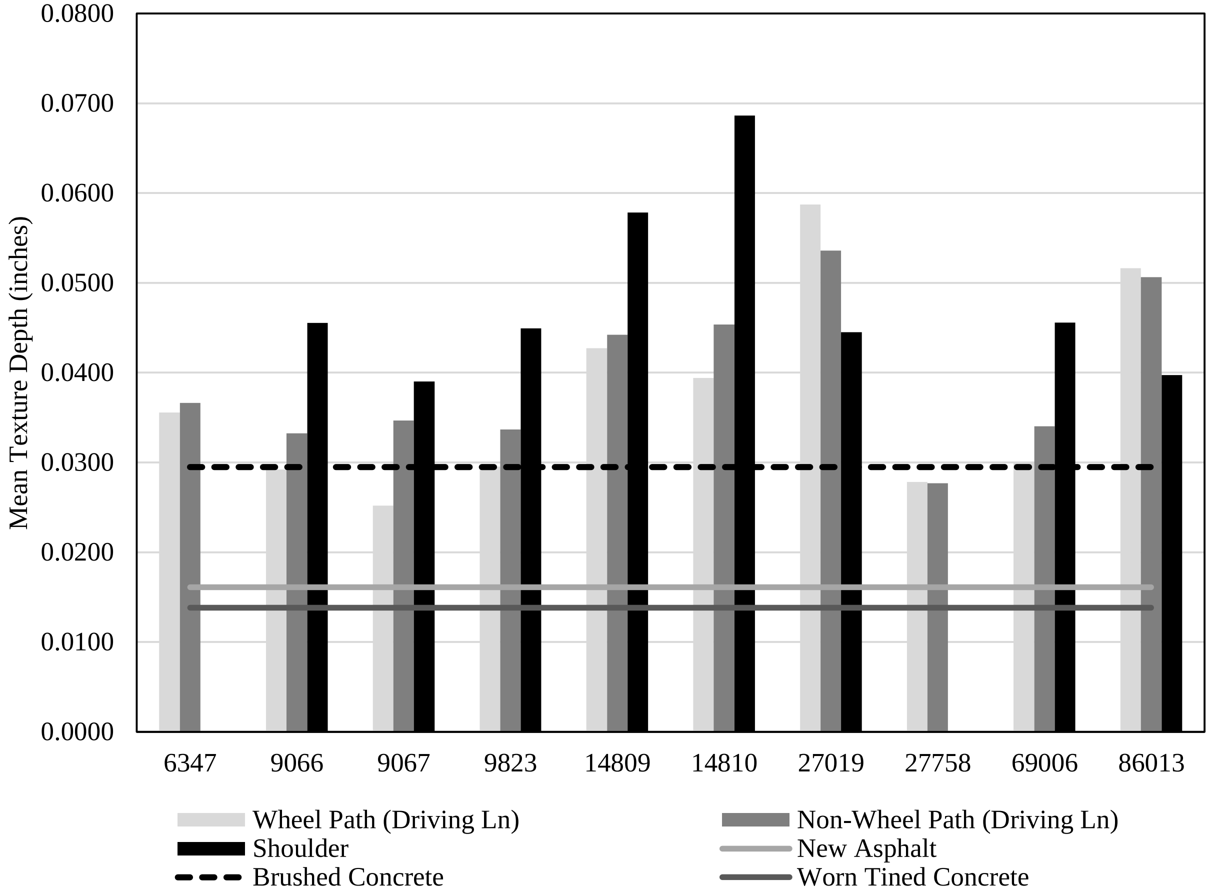


Figure 62: Mean Texture Depth (Year-2: Driving Lane)

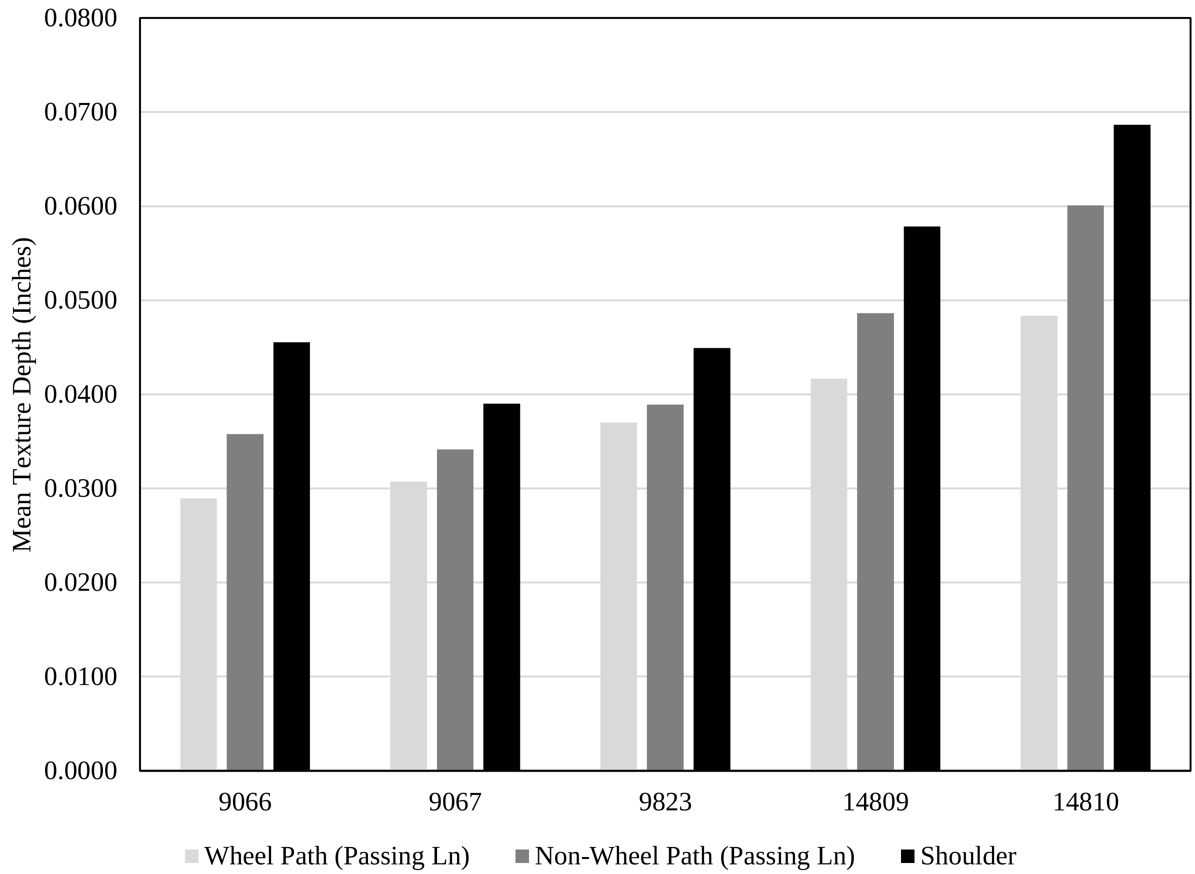


Figure 63: Mean Texture Depth (Year-2: Passing Lane)

The bridges with a Poly-Carb system (Br 9066, 9067, 9823, and 69006) or Transpo system (Br 6347, and 27758) installed all have mean texture depth values near or below that of brushed concrete. When comparing the driving lane to the shoulders it shows that these systems have lost a significant amount of texture and depth due to snowplow and traffic abrasion.

The Poly-Carb systems use smaller aggregates compared to Novachip and SafeLane systems; therefore Poly-Carb systems are more susceptible to a reduction in depth bringing the mean texture depth closer to traditionally paved roadway surfaces. However the SafeLane bridges (14809 and 14810) also had a significant reduction in depth when comparing their driving lanes and shoulders. With the assumption that little abrasion has occurred on shoulder lanes for Poly-Carb and SafeLane, an estimate can be made for the original MTD when the system was first installed and had no abrasion or distresses. Based on the shoulder depth, SafeLane systems have a starting MTD between 0.06 and 0.07 while Poly-Carb systems start with an MTD between 0.045 and 0.05. With these assumed starting numbers, the Poly-Carb and SafeLane systems have had a 30% to 35% reduction in MTD values. Using the dates of installation SafeLane has a reduction of about 5% per year. Poly-Carb has a reduction of 6%-8% a year. It should be noted that reduction in MTD values are greatly influenced by winter weather and the use of snowplows. From 2013 to 2014 (Figure 65), the percent of MTD reduction for bridges in this study which endured harsh winter conditions varied from 4%-20%. Transpo systems were excluded from this comparison because testing of the shoulders on bridges with a Transpo system was not conducted.

The difference between the MTD in the driving lane, passing lane, and shoulder for some Poly-Carb and SafeLane systems is illustrated below in Figure 64. In general the passing lane has a larger depth and it is reasonable to attribute this trend to higher traffic loads (especially truck loads) traveling in the driving lane.

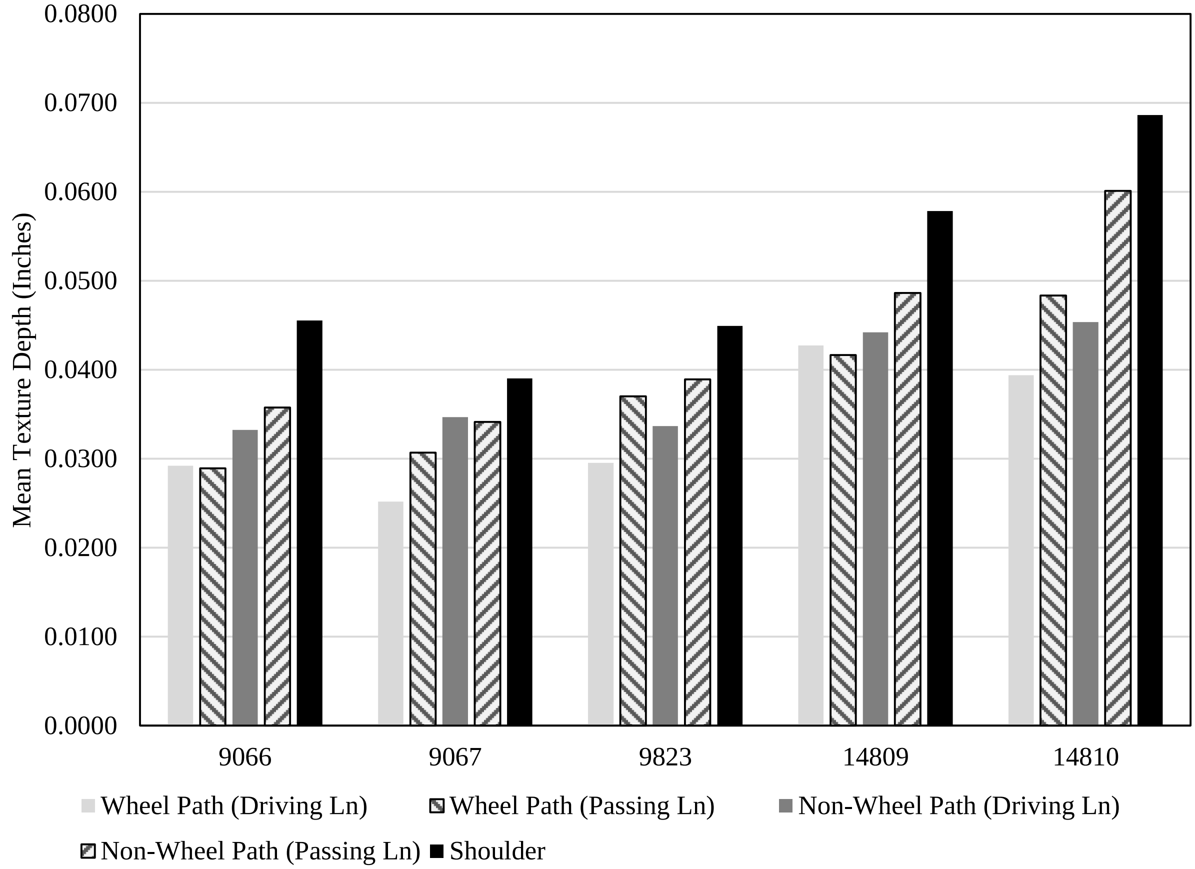


Figure 64: Mean Texture Depth Comparison

Five bridges were tested in both the first and second year of this study including the Poly-Carb bridges 9066, 9067, 9823, and 69006 as well as the SafeLane Bridge 14810. The comparison between the two years is shown in Figure 65. As expected, another year of abrasion has caused a reduction in depth. For Bridge 9066 (Moorhead, I-94 Westbound) the depth in the wheel path actually increased. As explained in the year one field observations, this bridge has several significantly polished areas and measurements of these areas were taken. Thus the 2013 values for this bridge targeted polished areas, whereas the 2014 values are more representative of the mean texture depth. Bridge 9067 (Moorhead, I-94 Eastbound) saw the most significant drop in depth from 2013 to 2014. Bridge 14810 (Barnesville, I-94 Eastbound) had significant abrasion to its non-wheel path reducing its average depth and bringing it much closer to the wheel path values. The

2013-2014 winter was very harsh which may have resulted in above average snow plow abrasion.

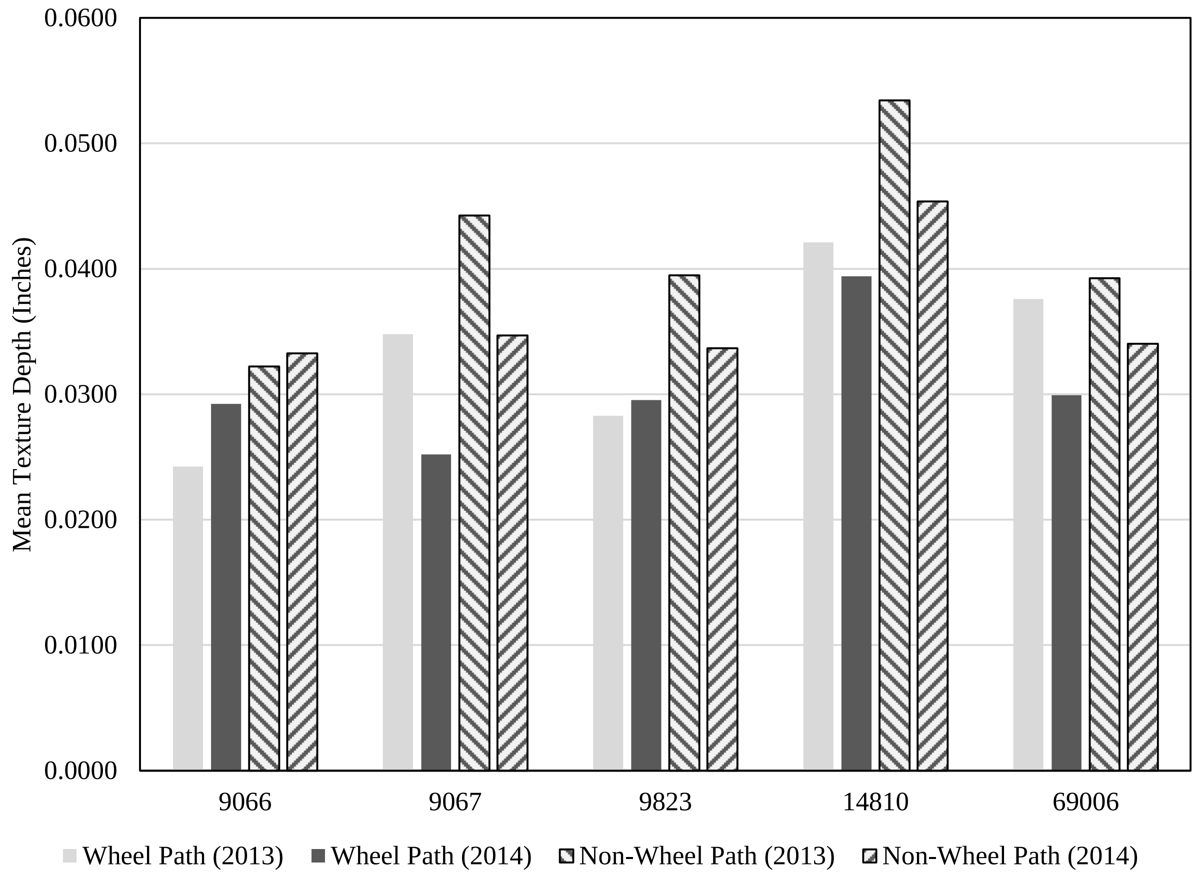


Figure 65: Mean Texture Depth Comparison of 2013 (Year-1) to 2014 (Year-2) Values

An estimation for the number of vehicles passing over each bridge was found using the average daily traffic (ADT) and number of years the HFO system has been in place. Using this data, the relative difference between the wheel and non-wheel path was found for every one million passing vehicles. This relative difference normalized to traffic is displayed in Figure 66. This data does not account for unequal lane or directional distributions. Additionally it assumes that every vehicle causes the same amount of abrasion to the bridge aggregates.

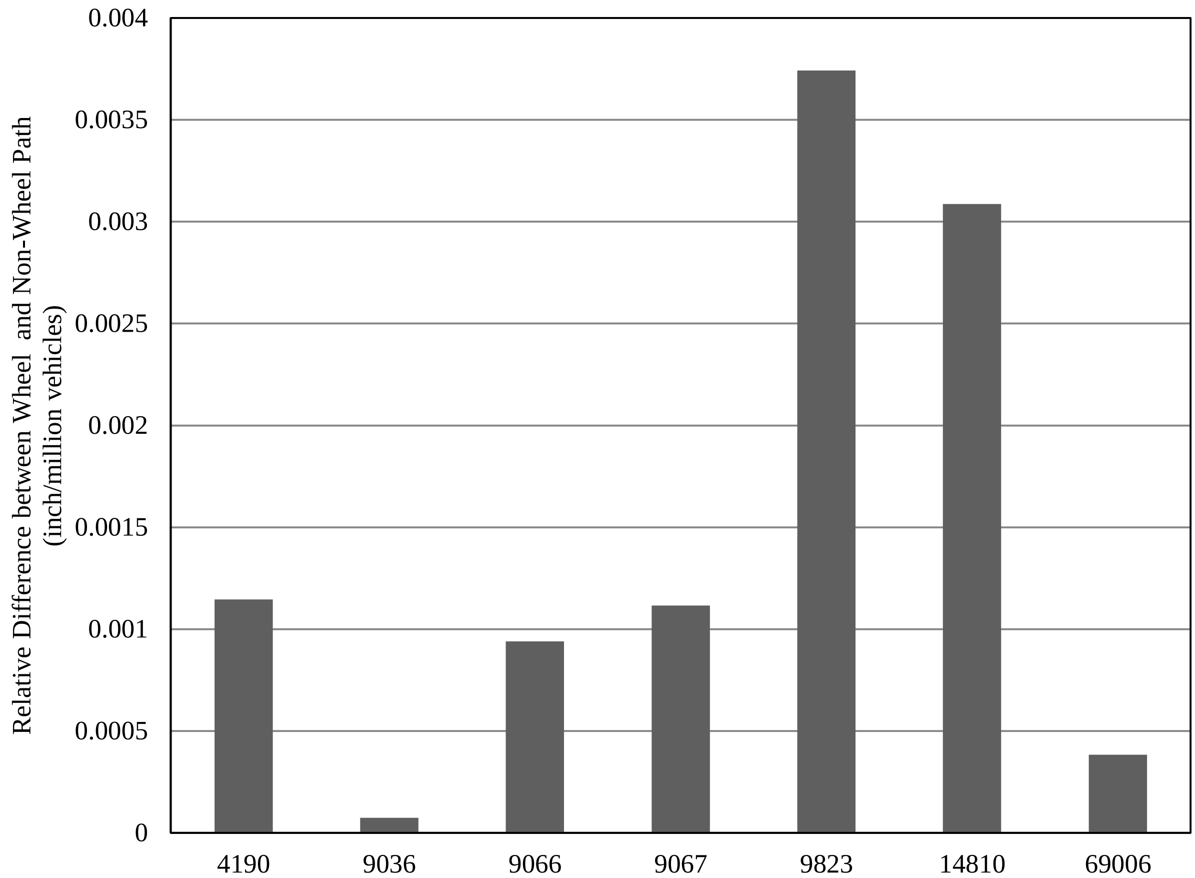


Figure 66: Relative Difference between Wheel and Non-Wheel Path MTD Normalized to Total Traffic

By comparing the relative difference between wheel and non-wheel MTD normalized to total traffic several conclusions are made:

- Taconite performs best against wheel path abrasion.
- Bridge 9823 (Atkinson, MN) and Bridge 14810 (Barnesville, MN) had significantly higher wheel abrasion compared to the other bridges.
- Channelized driving patterns on roadways with higher speeds, such as Bridge 9823 (Atkinson, MN) and Bridge 14810 (Barnesville, MN) are more apparent with the higher relative difference.

- An aggregate type does not wear evenly in every location. Considering basalt as an example, it clearly shows that different traffic loads, speeds, and conditions all affect the abrasion rate.

Overall several conclusions can be drawn based on the test results:

- The wheel path abrasion is significantly higher on bridges where channelized driving is common (high speeds or high ADT). This is apparent on the higher speed Interstate bridges (Br 9823, Br 9066, Br 9067, Br 14809 and Br 14810), and the higher ADT bridge (Br 4190)
- Taconite (Br 69006) appears to have the best performance of the aggregates for abrasion due to wheels.
- The small difference between non-wheel and wheel path depths for Br 9036 (Robert St in St. Paul, Poly-Carb) and Br 86013 (CSAH 36 in Otsego, Novachip) is due to the slower speed limit which limits the abrasion damage. Also, since this bridge is a local road, lane changes may be more frequent causing a more even abrasion across the entire lane.
- The smaller depth of Br 9066 compared to Br 9067 suggests that higher traffic levels travel in the westbound driving lane. This supports the claim that Bridge 9066 is functionally obsolete. Higher traffic in this lane is most likely due to the traffic exiting to the area Universities.
- Bridge 9066 (WB I-94 in Moorhead, Poly-Carb) and Bridge 9823 (SB I-35 in Atkinson, Poly-Carb) had the smallest reductions in mean texture depth from the year 2013 to 2014.

**Bond Strength Results**

The bond strength of the different HFO systems was tested during both years of the study. As previously mentioned in the Chapter 4 Testing Procedure and Methodology section, a slight variation in testing methods occurred between the years, and for that reason, torque values from Year-1 should not be compared to the values found in Year-2.

***Year-1 Results***

The torque bond test was conducted on four bridges. Some bridges visited during Year-1 were not tested due to time restrictions with traffic control. The results for the testing conducted are presented in Figure 67.

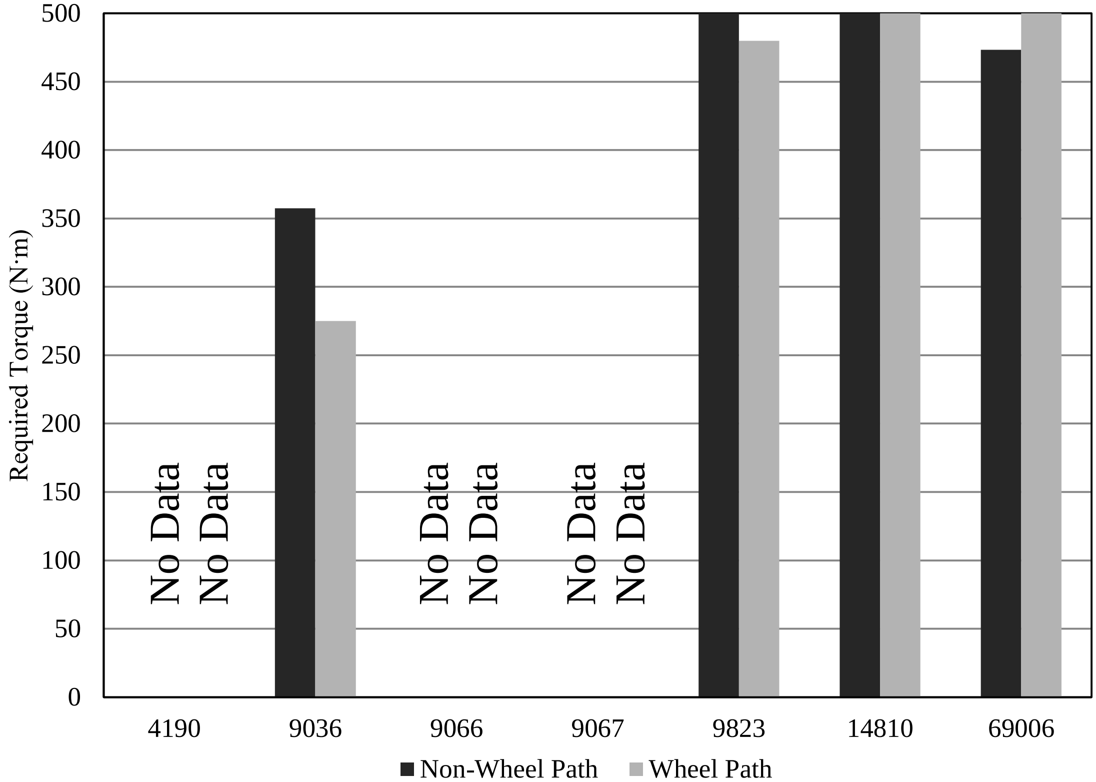


Figure 67: Year-1 Torque Test Results

Three of the four bridges tested recorded a torque strength of around 500 N·m. Although the strength on Bridge 9036 (Robert St) was lower, only one of the tests removed the epoxy system from the bridge deck. All other tests performed on Robert Street broke the epoxy securing the steel plate to the system. Weather conditions on the day of testing are most likely the cause for this lower epoxy strength. The conditions were sunny with temperatures above 90°F [32°C]. With even higher roadway surface temperatures, the epoxy used to secure the steel plate was reaching its upper operating temperature limit. Two tests removed the epoxy system from the bridge deck.

- Bridge 9036 (Robert St): Non-wheel path test removed the system with a torque of 210 N·m. This test was the test performed closest to the damaged section of the system. This indicates that the debonding of the system may slowly be spreading away from the already damaged area.
- Bridge 14810 (Barnesville, MN): Non-wheel path test removed the system with a torque of 500 N·m [370 ft·lb]. The system was removed after the strength gauge on the torque wrench was maxed out.

***Year-2 Results***

A more robust analysis of the bond strength was completed during the second year of the study. The torque bond test was conducted on eight bridges during the year two study. Bond strength results varied greatly compared to results from the first year of study, but the variation is most likely from the small change in the testing procedure. In the second year, the required torque for removal of the steel plates was much lower and the removal of the HFO system was more common. In the first year, excluding Bridge 9036 because it is considered an additional bridge, one of the eighteen tests conducted on three different bridges removed the HFO system from the bridge deck. In Year-2, 30 of the 53 tests conducted on eight bridges removed the HFO system from the bridge deck. A varying degree of removal occurred. This various degree of removal is illustrated below in Figure 68 and is quantified by “no removal” (left), “partial removal” (middle), and “total removal” (right).

Figure 68: Various Degree of Removal Including "No Removal" (Left), "Partial Removal" (Middle), and "Total Removal" (Right)

The results for the testing conducted are displayed in Figure 69 in the unit of Newton-meters. From the data below it is hard to find a conclusive general trend for the strength of systems.

Torque for bridges with the same systems have similar torque values.

- Novachip systems varied from 100 N·m to 150 N·m [74 to 111 ft·lb]
- SafeLane systems varied from 150 N·m to 200 N·m [111 to 148 ft·lb]
- Poly-Carb systems varied from 175 N·m to 450 N·m [129 to 332 ft·lb]
- Transpo systems varied from 120 N·m to 420 N·m [89 to 310 ft·lb]

Poly-Carb and SafeLane are epoxy overlay systems while Novachip is a thin-bonded open graded asphalt overlay. With epoxy having better tensile strength properties compared to asphalt, Novachip has the smallest required torque when internal tensile stresses are placed on the HFO systems as a result of the torque placed on the system.

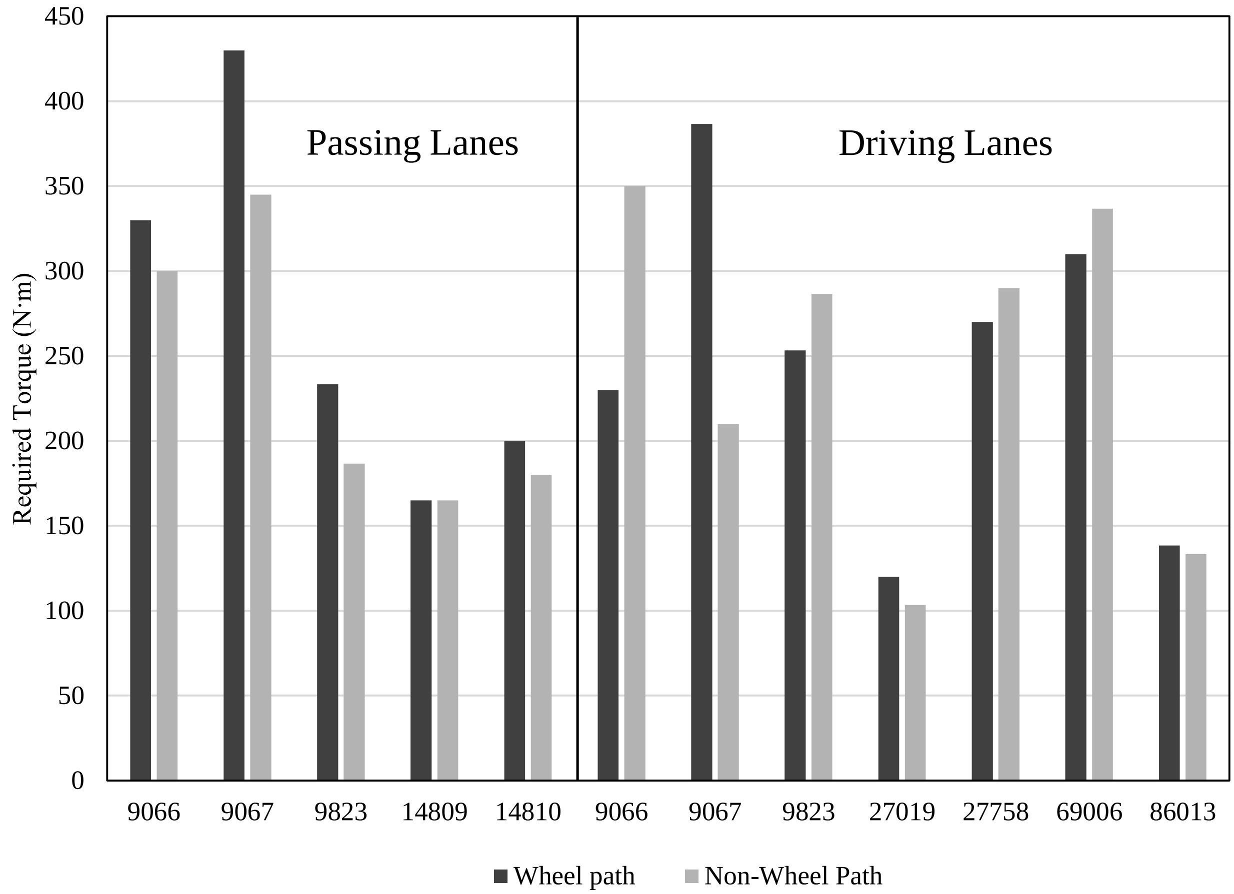


Figure 69: Torque Bond Test Results

Values from the non-wheel and wheel path are similar for each individual bridge with a variance of 50 N·m [37 ft·lb]. When comparing the difference between passing lane and driving lane values that variance increases to 75 N·m [55 ft·lb]. Bridges 9066 and 9067 (I-94 in Moorhead, Poly-Carb) have a Poly-Carb system with Flint aggregate. These bridges had an average variance of 100 N·m [74 ft·lb] between non-wheel and wheel path. The flint aggregate particles are the smallest used in the HFO systems in this study and tend to be more flat compared to aggregates in other systems. During installation of systems with flint, it is possible to have many of the flat particles placed with similar orientation. This orientation and small particle size reduces the amount of aggregate interlock and can cause a weak plane in the system. This is illustrated below in Figure 70. The figure shows two samples that were removed using the torque bond test. On the left is the partial removal of a Poly-Carb system with Flint aggregates (Br 9067) while the sample on the right is a partial removal of a SafeLane system (Br 14810). The partial removal of flint happened in a horizontal plane where the SafeLane partial removal depth varies across the sample.

Figure 70: Comparison of Poly-Carb with Flint (Left) and SafeLane Torque Removal Samples

In the driving lane, both of the bridges with a Novachip system had 100% of the tests with at least a partial removal of the HFO system. The only other removals in the driving lane occurred in the wheel path of Bridge 9066 (WB I-94 in Moorhead, Poly-Carb) and Bridge 69006 (TH 53 in Virginia, Poly-Carb) (Figure 71).

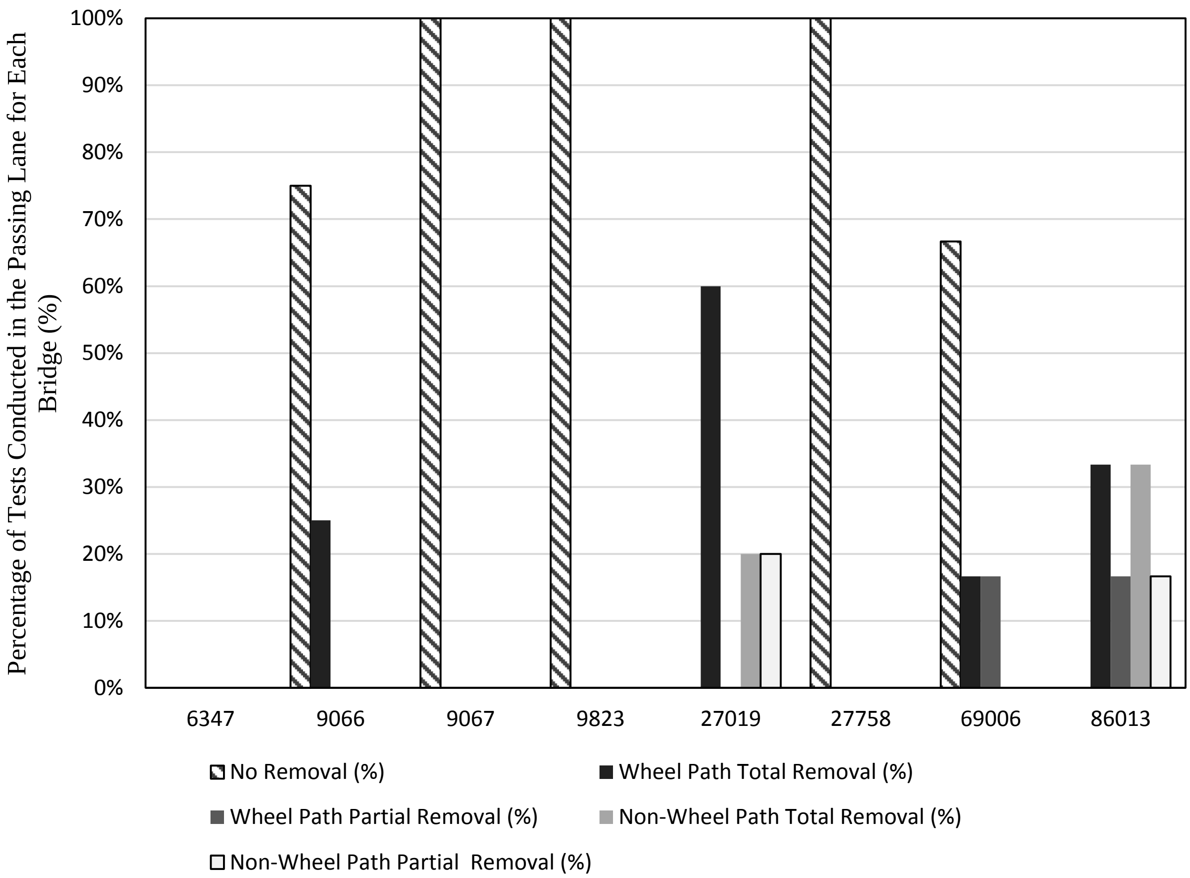


Figure 71: Percentage of Tests in the Driving Lane Removing HFO Systems from the Bridge Deck

The removal rate of the HFO systems in passing lanes during torque testing is provided in Figure 72. Of the five passing lanes tested, four had 100% of the tests remove at least a portion of the HFO system. The other bridge (Br 9823, TH 53 in Virginia, Poly-Carb) did not have any system removal in the passing lane. There are not any apparent conditions that point to why removals are more common in the passing lane compared to the driving lane. One possibility is the variation of construction. As previously discussed, visual inspection can often reveal different distress and abrasion patterns on different sides of construction joints. This variability may cause the variability in torque testing both laterally and transversely along the bridge.

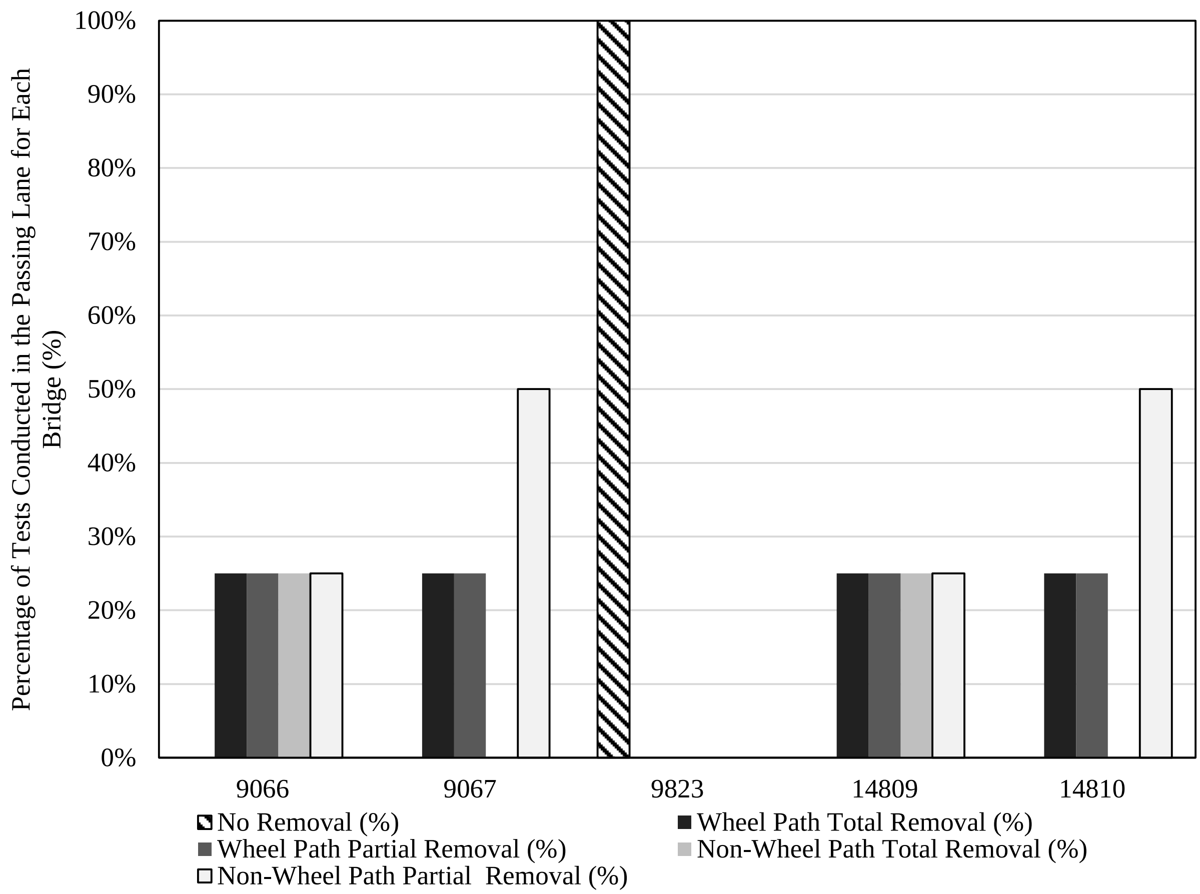


Figure 72: Percentage of Tests in the Passing Lane Removing HFO Systems from the Bridge Deck

**Permeability Results**

The permeability factor found during all testing was very small and for practical purposes may be considered zero. The macrotexture that exists from the aggregate used in these overlay systems creates void space between some aggregate particles. Water slowly infiltrates these void pockets throughout the duration of the test. This infiltration is the reason behind permeability coefficients (K values) that are very small but not zero. For the SafeLane (Br 14809 and Br 14810) and the Novachip (Br 27019 and Br 86013) systems, the aggregates were too large to allow for permeability testing. The large aggregate on the surface of these systems prevented a strong and complete seal when attaching the testing equipment to the roadway. Weather prevented testing from occurring for both Transpo bridges. Results are presented in Table 19.

Table 19: Permeability Results

| Bridge | Coefficient of Permeability (K) | | | |
|---|---|---|---|---|
| | Year-1 | | Year-2 | |
| | Wheel Path (cm/sec) | Non-Wheel Path (cm/sec) | Wheel Path (cm/sec) | Non-Wheel Path (cm/sec) |
| 6347 | N/A | N/A | N/A | N/A |
| 9036 | $2.26x10^{-7}$ | $1.855x10^{-7}$ | N/A | N/A |
| 9066 | 0 | $2.102x10^{-7}$ | $2.19x10^{-7}$ | $2.32x10^{-7}$ |
| 9067 | $1.779x10^{-7}$ | N/A | 0 | $2.28x10^{-7}$ |
| 9823 | $9.046x10^{-7}$ | $2.016x10^{-7}$ | 0 | $2.34x10^{-7}$ |
| 14809 | N/A | N/A | N/A | N/A |
| 14810 | $3.811x10^{-6}$ | N/A | N/A | N/A |
| 27019 | N/A | N/A | N/A | N/A |
| 27758 | N/A | N/A | N/A | N/A |
| 69006 | 0 | $1.007x10^{-6}$ | $4.39x10^{-7}$ | 0 |
| 86013 | $9.814x10^{-5}$ | $5.520x10^{-6}$ | N/A | N/A |
| 86019 | N/A | N/A | N/A | N/A |

This test was also conducted on a traditional concrete bridge deck to provide a benchmark for comparison to bridges without an HFO system. In Year-2 this test was also conducted on Bridge 9824, which is the counterpart for Bridge 9823 in Atkinson, MN. Bridge 9824 does not have an HFO system; it has a tined concrete bridge deck. The test was conducted on the bridge deck in two locations. The first did not have any distresses and resulted with a K value of 0 cm/s. The second test was over an unsealed crack in the bridge deck (Figure 73) with a resulting K value of $1.53 \times 10^{-6}$ cm/s. Some Department of Transportation agencies have permeability criteria for asphalt pavements with an acceptable range between 0 and $125 \times 10^{-5}$ cm/s (Williams, 2009). Table 20 displays the ranges discussed to provide a visual of the differences between the values.

Table 20: Coefficient of Permeability Ranges for Different Paved Surfaces

| Different Paved Surfaces | Coefficient of Permeability (cm/s) |
| --- | --- |
| Acceptable Range for Asphalt pavement | 0 to $125 \times 10^{-5}$ |
| Concrete with Crack (Br 9824) | 0 to $1.53 \times 10^{-6}$ |
| HFO systems | 0 to $4.39 \times 10^{-7}$ |

Figure 73: Area of Bridge 9824 Tested for Permeability

HFO systems provide a much more impervious surface which is one of their design benefits. This impervious barrier helps to protect bridge decks from harmful deterioration and corrosion from chloride penetration.

The aggregates used for the Novachip systems were too large to allow a proper seal for permeability testing. A test was conducted on the shoulder where high abrasion had reduced the macro texture. During this test it appeared that a proper seal was achieved and some water was permeating into the system. After a short duration of time (one or two minutes) water would percolate back to the surface from outside the diameter of the testing apparatus. This suggests that water may be infiltrating into the system and moving laterally however it is impossible to say at what rate and how far into the system water is able to infiltrate.

## Surface Friction Results

This section discusses the testing conducted on the surface of HFO systems. Results of testing provide skid numbers or the coefficient of friction for a given surface. A discussion of skid numbers is found above in Chapter 2.

### *Surface Friction Using a Skid Trailer*

A skid trailer provides surface friction values in the form of skid numbers, which were provided by MnDOT. The bridge deck skid numbers are plotted against time in Figure 74 (ribbed tire) and Figure 75 (smooth tire). Figure 76 is the average skid numbers for Bridge 14810 and is provided as an example in this section. Some bridge decks were reported to have too much friction to properly test. This is represented in the plots with year one skid numbers that are larger than 100. The ranges found for PCC pavements are also in Figure 74 and Figure 75. They are labeled Turf Drag PCC and appear as an area with cross-hatching. It should be noted that the dashed portions of the lines in the following figures indicate an approximation based on the available data.

The SN values in Figure 74 and Figure 75 show that several HFO systems are still above a SN of 50 several years after installation, while others have fallen below 50 and into the "potential action segment" threshold.

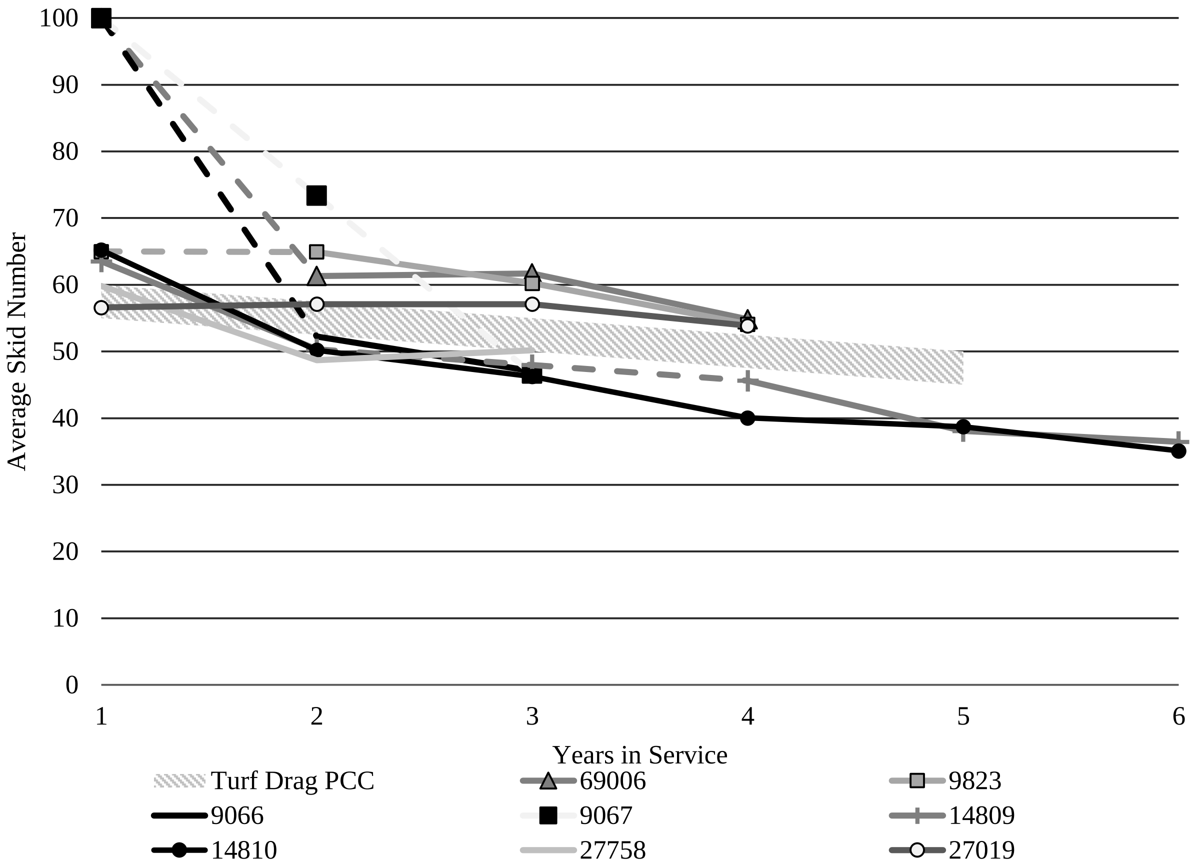


Figure 74: Average Ribbed Tire Skid Number over Time

In general the first year of service has a significantly larger loss compared to subsequent years. After this first year the slopes for the different bridges are approximately the same. This means the friction of all systems is degrading at a similar rate. Abrasion is more likely to occur with particles that have substantial angularity. As the edges of angular aggregates are rounded over time due to abrasion the rate at which they degrade significantly decreases. Aggregate particles not fully bonded with epoxy matrix may be removed with relative ease. The Novachip system (BR 27019) is the only bridge that did not have a significant reduction in its skid number over the first year of service. Comparison to normal pavement skid numbers is possible by comparing these systems to the skid numbers of Turf Drag PCC. Of the eight bridges tested, seven of them are above the Turf Drag PCC range while the other bridge (Br 27019) is within the range. However after three years, five of the bridges are at or below the bottom of this range. Interestingly the Novachip system (Br 27019) is the only system inside the range at year one, but is one of the three bridges above the range after the third year. The fact that these systems approach this range within three to five years reveals that HFO systems only have comparatively higher friction values for a short period of time in regions that rely heavily on snow plows. Severe winter conditions and maintenance common in Minnesota cause abrasion that rapidly reduces the life of a high friction system. This does not suggest that systems are by any means unsafe, but that skid numbers quickly become comparable to that of a traditional bridge deck.

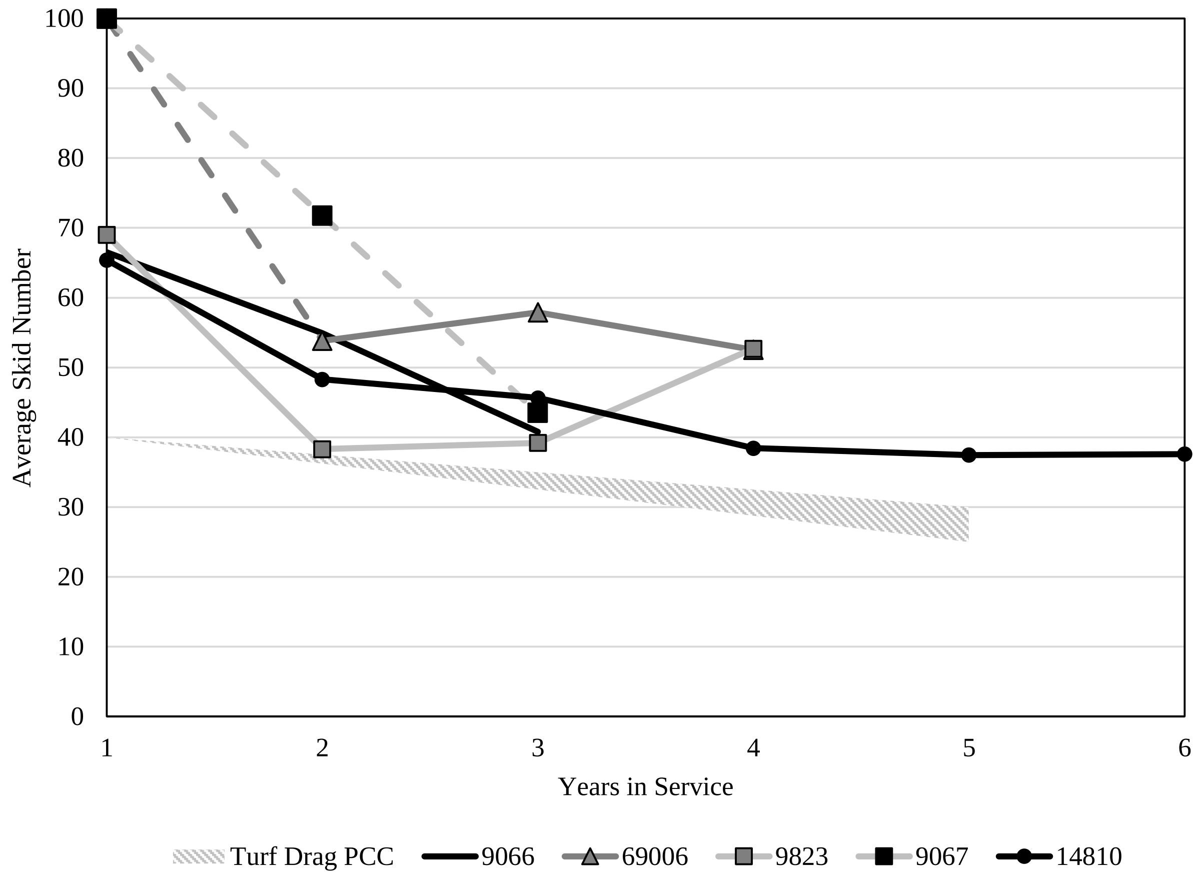


Figure 75: Average Smooth Tire Skid Number over Time

On the smooth tire plot some of the slopes have a positive slope. This is evidence that the smooth tire is sensitive to water film depth. The large macrotexture in the epoxy systems have the capacity to hold more water which may cause the variance in results. The bridge decks all stay above the range of Turf Drag PCC. This suggests that the microtexture of the epoxy systems are better than that of a PCC pavement.

Bridge 14809 and 14810 (Barnesville, MN) have skid numbers for the bridge deck before and after the installation of the epoxy system in 2007. A major reason for the installation of these epoxy systems is the improved friction in poor weather condition. This information is useful to analyze how long the HFO system provides a system with better skid numbers higher than conditions without such a system. Figure 76 is the average skid numbers for Bridge 14810. As shown the skid numbers for the bridge deck before the installation of the SafeLane epoxy system were 44.3 (ribbed) and 34.9 (smooth). The latest test conducted on July 24, 2012 resulted with skid numbers of 35.1 (ribbed) and 37.6 (smooth). In the five years since the placement of the system the bridge deck skid numbers have reduced to skid numbers similar to a bridge deck without an HFO system in place.

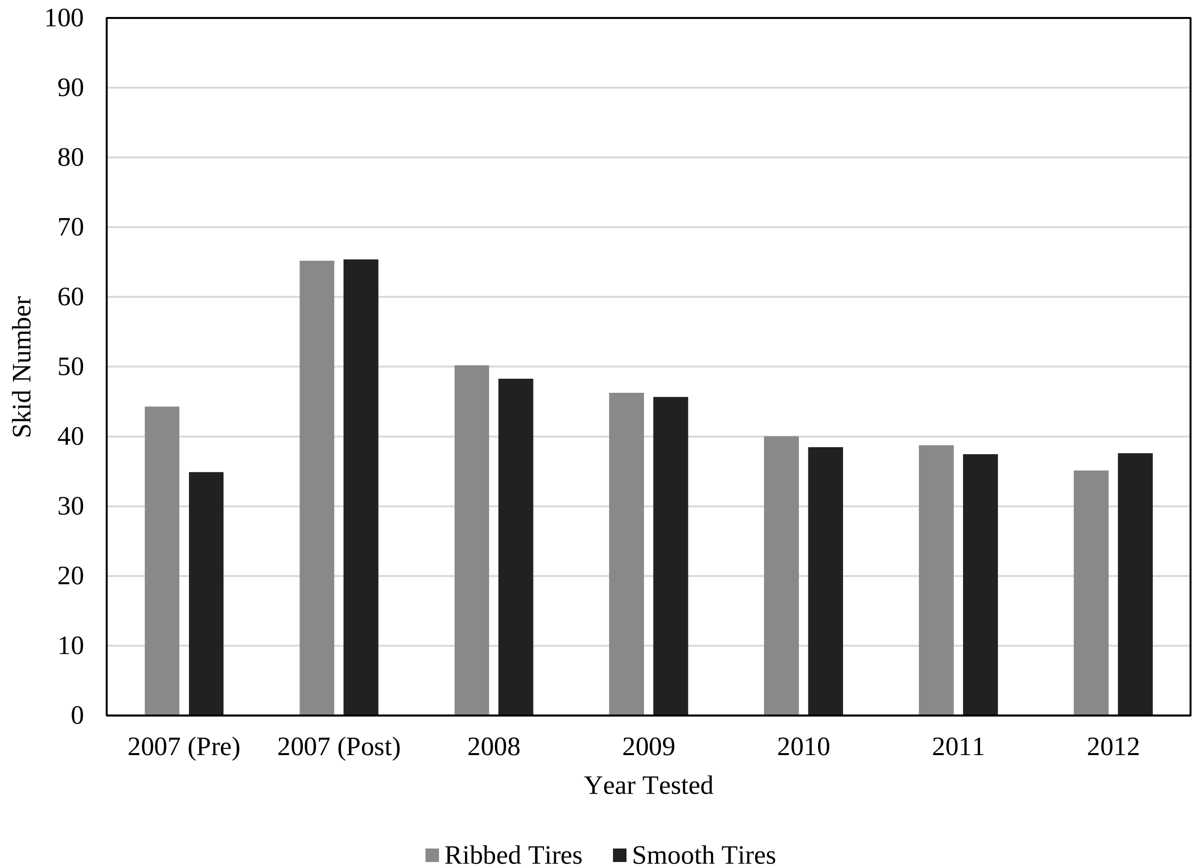


Figure 76: Bridge 14810 (Barnesville, MN) Average Skid Numbers

In Year-2 the skid numbers for Bridges 6437 (Osceola), 86013 (Otsego), and 86019 (Otsego) were provided (Figure 77) but there is not any previous data to compare with. The Poly-Carb systems' (9066 and 9067, I-94 Moorhead) skid numbers continue to diminish. The Novachip system's (Bridge 27019 TH 101 Otsego) skid numbers remained the same from 2012 to 2013. The Novachip system is the only bridge that did not have a significant reduction in its skid number over the first year of service, and its skid numbers reduce at a slower rate. This is due in large to the fact that Novachip is asphalt based and as explained previously has more elastic properties. This elasticity preserves the systems aggregates which in turn provides better friction values.

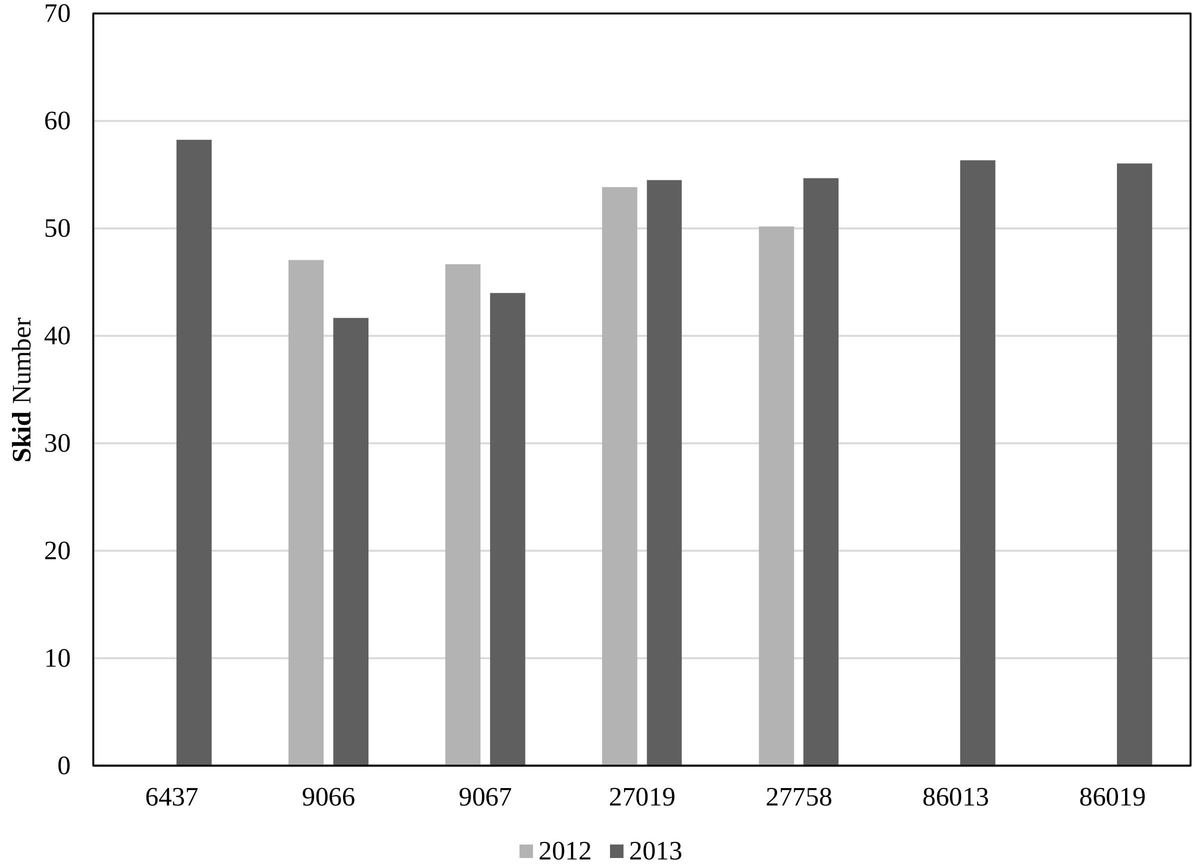


Figure 77: Skid Number Comparison (2012 vs 2013)

Since 2011 the skid numbers for Bridge 27758 (Penn Ave in Minneapolis, Transpo) have increased from 48.7 to 54.7 (Figure 78). Originally the rise from 48.7 in 2011 to 50.4 in 2012 was written off as either an error in testing or testing a different section of the bridge with better friction values. However it is now evident with the continual increase that it cannot be attributed to an error in the testing procedure.

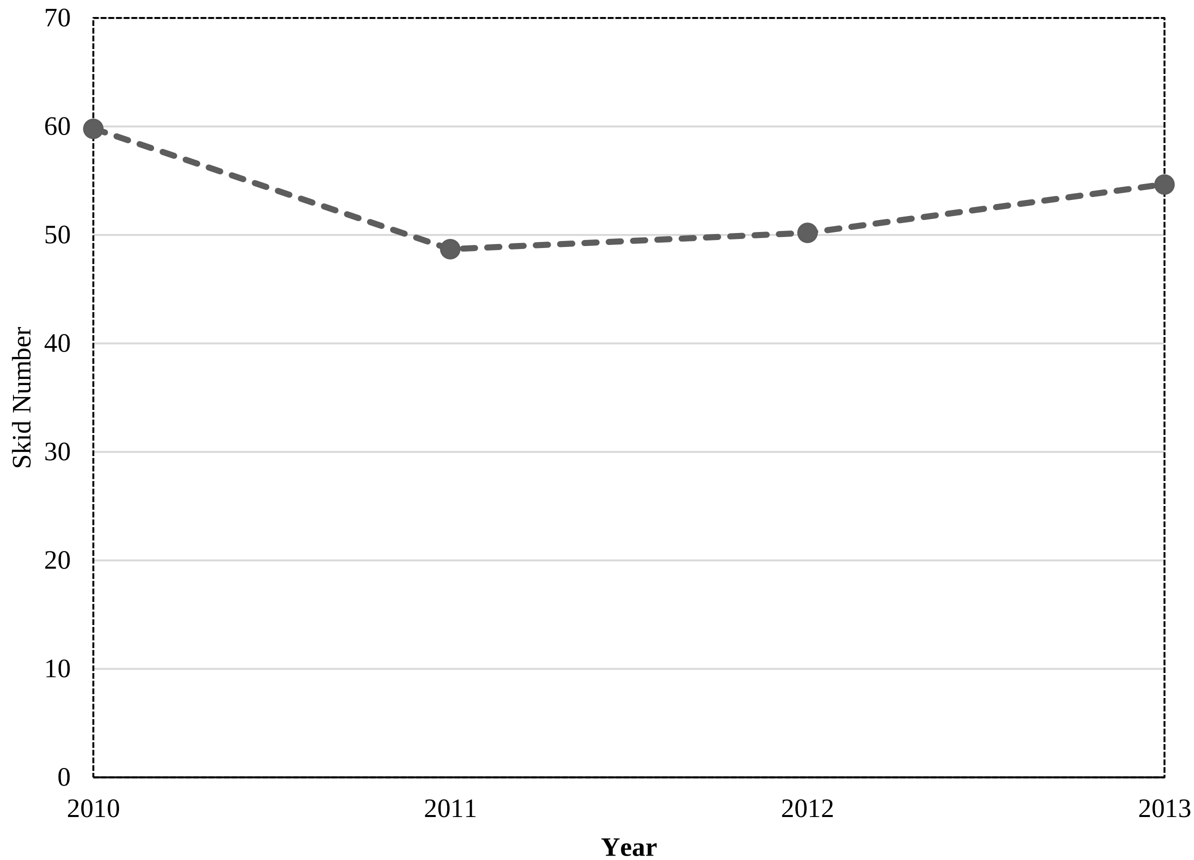


Figure 78: Skid Numbers for Bridge 27758 (Penn Ave, Minneapolis)

The HFO system placed on Bridge 27758 is very thin and as described earlier (Observation Section), the system is being removed completely in thin strips exposing the bridge deck. As parts of the bridge deck are being exposed the mean texture depth (MTD) is actually increasing. In an area of the system that is not distressed the MTD is from the top of the aggregate layer to where the aggregates are embedded into the epoxy. In a distressed are the MTD is from the top of the aggregates to the top of the concrete deck. Since the removed areas are small relative to contact area of a tire, the distress is actually increasing the MTD and improving the skid number of the system. This is illustrated in Figure 79.

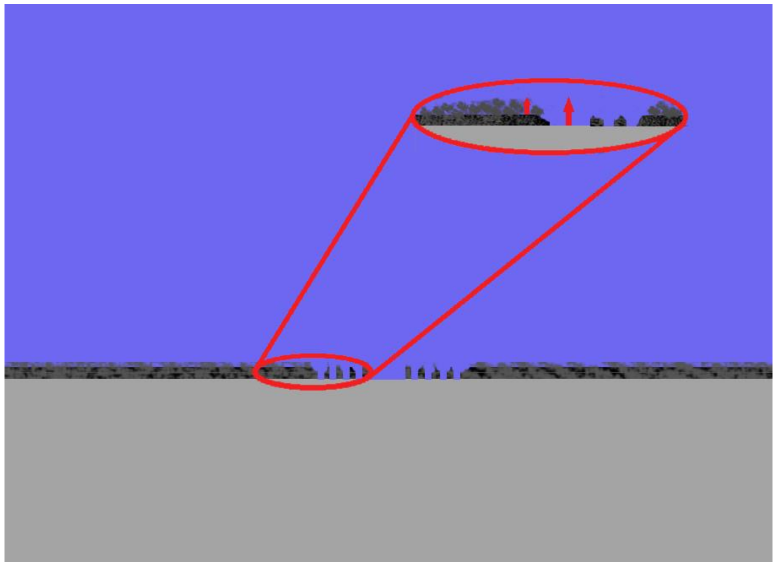

Figure 79: Distresses Helping to Increase the MTD and Skid Numbers on Bridge 27758

***Surface Friction Using a Dynamic Friction Tester***

The surface friction values are provided by the Dynamic Friction Tester (DFT) as the coefficient of friction. Each test conducted generated a report with values for the coefficient of friction at 0, 20, 40, 60, and 80 km/h (0, 12.4, 24.9, 37.3, 49.7 mph) as well as a chart graphing the coefficient of friction throughout the duration of the test (Figure 80).

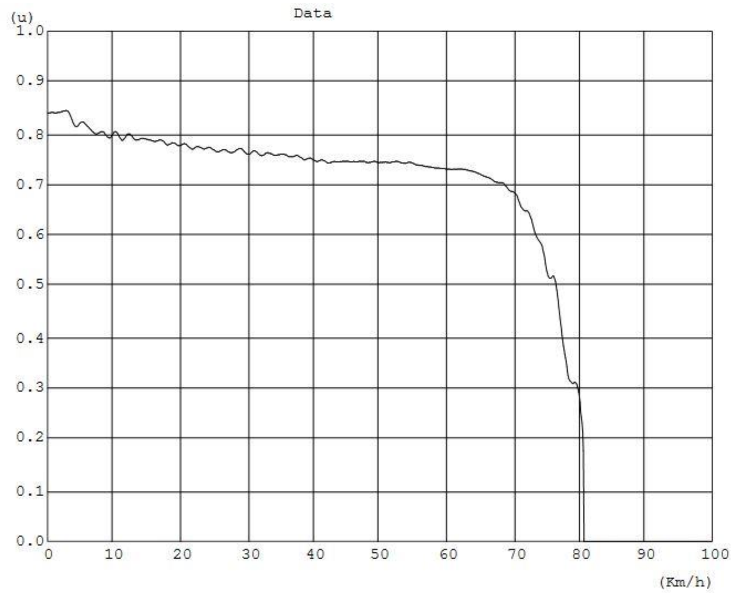


Figure 80: Coefficient of Friction throughout Duration of DFT Testing

Below are the average values for the coefficient of friction at 60 km/h for the wheel and non-wheel path of each bridge (Figure 81). For Bridge 14809 (WB I-94, Barnesville, SafeLane) data for the passing lane was lost in the field. The passing lane for Bridge 9823 (SB I-35 in Atkinson, Poly-Carb) was not tested. In general the wheel path has lower friction values compared to the non-wheel path. On average there is a 17% difference between the wheel and non-wheel path. Also, the driving lane has lower friction values compared to the passing lane with an average difference of 10%. The trends match up with the mean texture depth (MTD) trends discussed earlier. Higher abrasion (in the wheel path compared to the non-wheel path, and the driving lane compared to the passing lane) reduces the MTD, a reduction in the texture results in a reduction of friction. This correlation will be studied more and expanded upon in the upcoming final report.

There is very little difference between the driving and passing lane values for Bridge 9066 (WB I-94 in Moorhead, Poly-Carb) which matches the visual observation that this bridge was heavily worn with a lot of aggregate polishing present. Both the driving lane wheel and non-wheel path values are lower than any values in the passing lane on Bridge 9823 (SB I-35 in Atkinson, Poly-Carb). This bridge has a negative grade and curves horizontally to the right. When a snowplow drives over this particular geometry the snowplow's blade places a lot of pressure on the right side of the bridge.

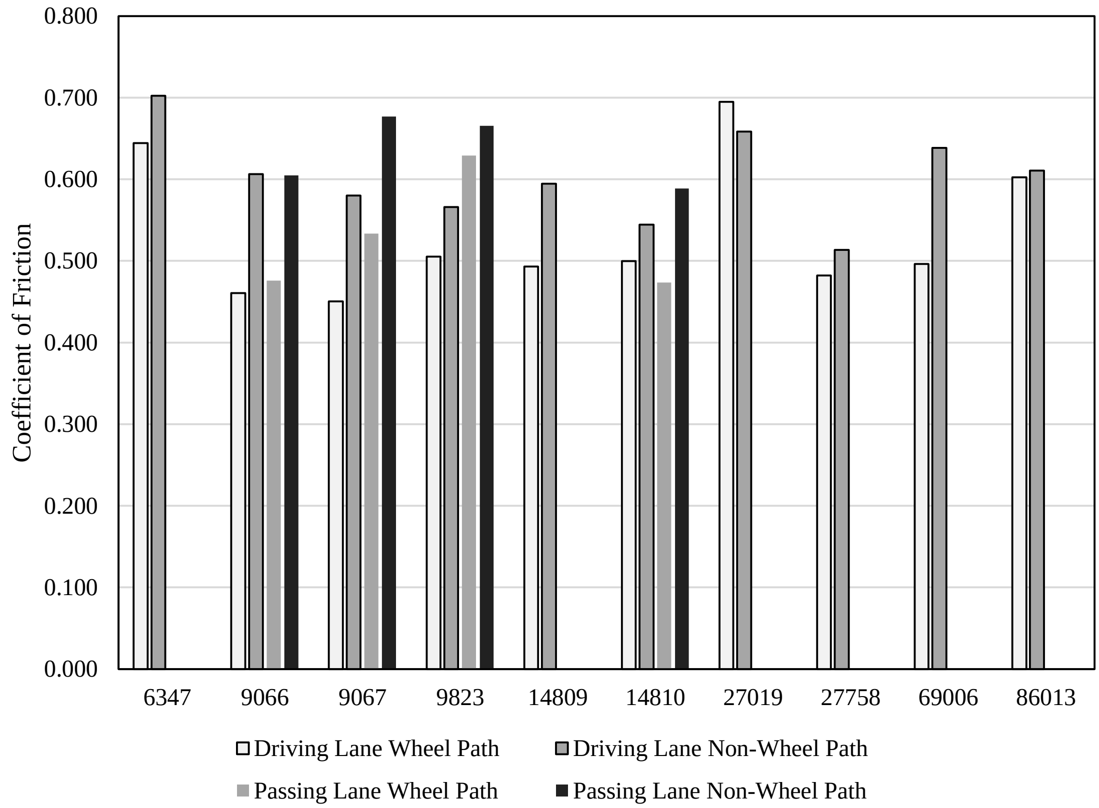


Figure 81: Average Coefficient of Friction at 60 km/h for Wheel and Non-Wheel Paths Results from Dynamic Friction Tester Results

### *Comparison of Dynamic Friction Tester and Skid Trailer Values*

Since the skid numbers found using the skid trailer are the coefficient of friction multiplied by 100, dividing the SN values by 100 allows for the comparison between the DFT and skid trailer values. Below in Figure 82 the coefficient of friction from both the skid trailer and DFT are plotted. It should be noted that the skid trailer testing is conducted at 30 mph (48.2 km/h) and the DFT plots values at 24.9 mph (40 km/h) and 37.3 mph (60 km/h).

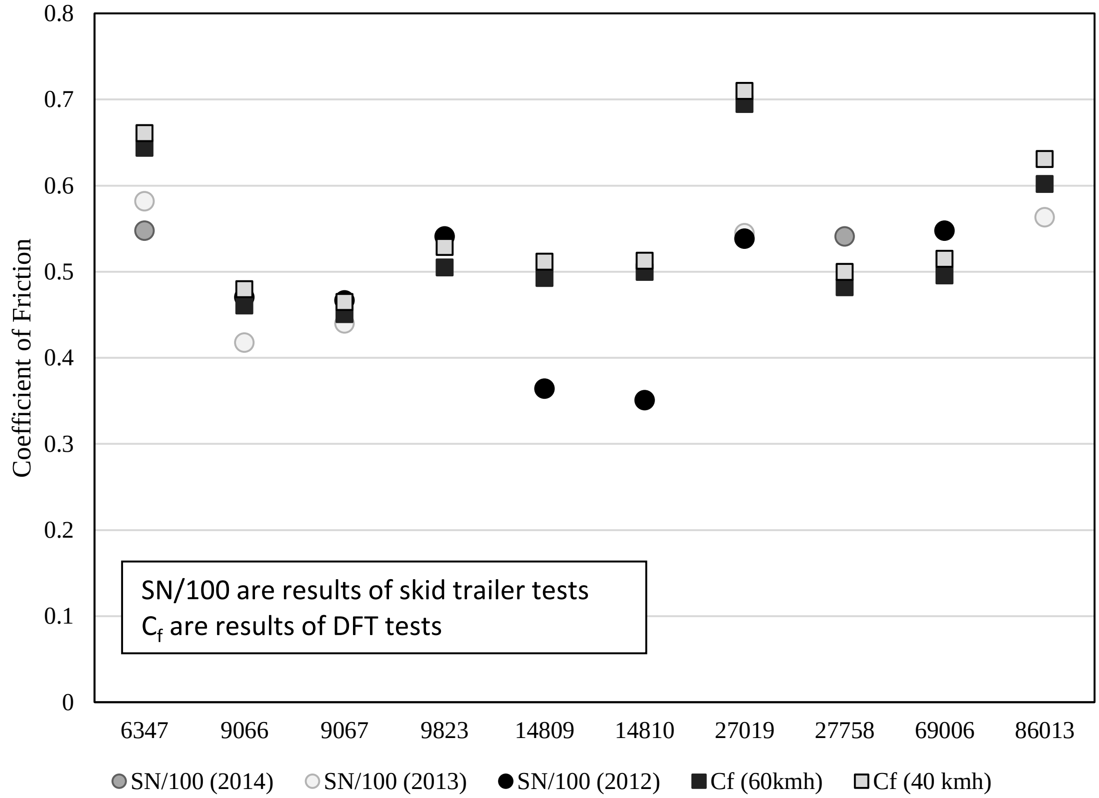


Figure 82: Comparison of Coefficient of Friction Found using the Skid Trailer and DFT.

For most of the bridges the coefficient of friction found using the skid trailer and the DFT are similar. The values vary the most in Bridges 6347, 14809, 14810, and 27019. Bridges 14809 (SafeLane), 14810 (SafeLane), and 27019 (Novachip) have HFO systems that use larger aggregates. This larger macrotexture may prevent the skid trailer from properly testing the surface. The larger aggregates sit higher out of the epoxy or asphalt they are embedded in. When a wheel drives across the surface it is only in contact with the tops of the aggregate. This reduced area requires less horizontal force and thus produces lower coefficients of friction. Bridge 6347 (Transpo), as previously discussed, had excess epoxy placed during construction and thus has smooth spots. A DFT test was run specifically on a smooth location resulting in a coefficient of 0.58 (40 km/h) and 0.59 (60km/h), which are significantly lower than the average DFT values for these speeds. Thus when a skid trailer is run over a longitudinal section of the roadway to obtain the coefficient of friction, it combines these smooth areas and normal areas resulting in a reduced coefficient of friction.

Overall the DFT and skid trailer tests provide similar surface friction values. It is recommended that DFT testing becomes a more prevalent test method because of the benefits it provides compared to the skid trailer. It allows for multiple tests run in the same location providing higher precision. It also eliminates error that may interfere with test results such as bouncing, lateral movement, improper water spray film thickness, or incorrect speeds of the skid trailer which must be in motion for its testing procedure. The fact that testing is done in a static location for DFT testing provides the opportunity to test more accurately in wheel and non-wheel paths. It also provides the opportunity to test specific distressed areas to compare with non-distressed areas.

### *Comparison of the Coefficient of Friction to the Age of an HFO System*

The coefficient of friction results from DFT testing were compared against the number of years each system has been in service (Figure 83). This comparison will provide information about the retention of surface friction over time. Similar comparisons were made above mapping the skid numbers of systems over time, however with only one year of DFT data this is not possible.

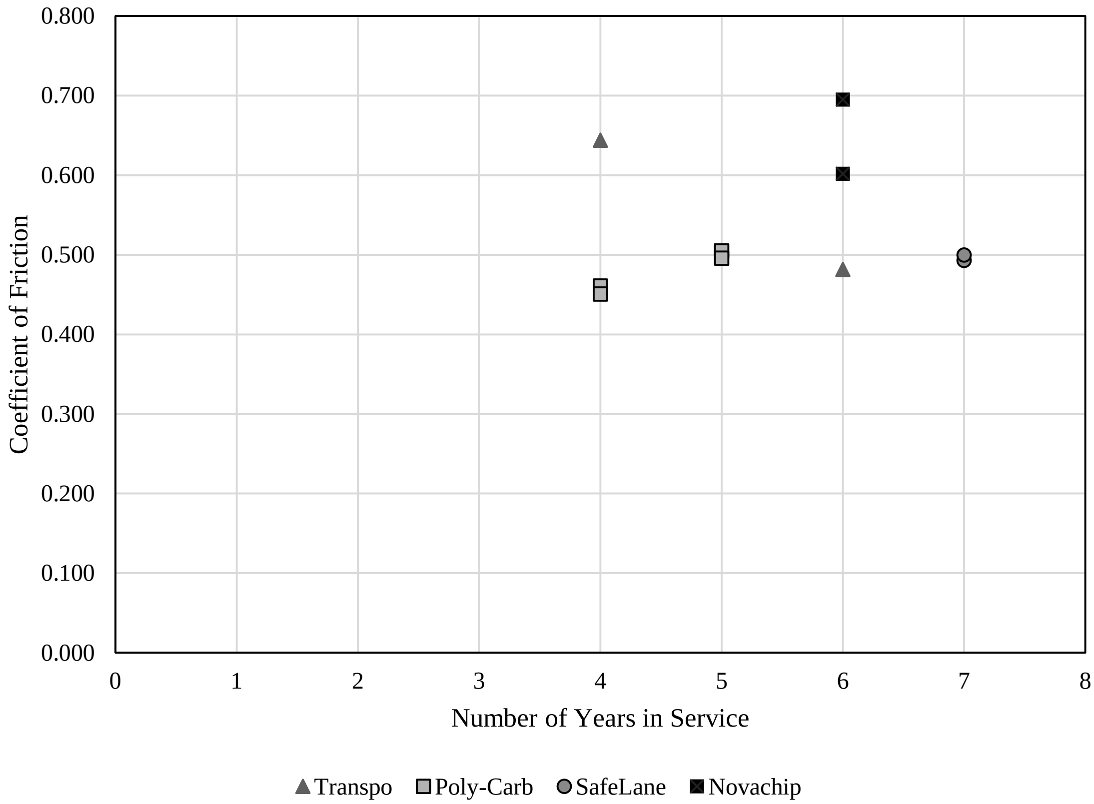


Figure 83: Coefficient of Friction versus the Number of Years in Service

As shown by this data the different epoxy systems reach the coefficient of friction of 0.5 (equivalent of a 50 SN value) between the third and seventh year of service. The Novachip system however averaged 0.65 after six years in service. Novachip systems are able to maintain a higher coefficient of friction for a longer time period compared to epoxy systems. This is due to the fact that aggregate raveling is not a common distress with Novachip systems and they are able to retain their aggregates.

***Comparison of the Coefficient of Friction to the Mean Texture Depth***

The coefficient of friction results from DFT testing were compared with the mean texture depth (MTD) results from the sand patch testing. The comparison shows that as MTD increases, so does the coefficient of friction (Figure 84).

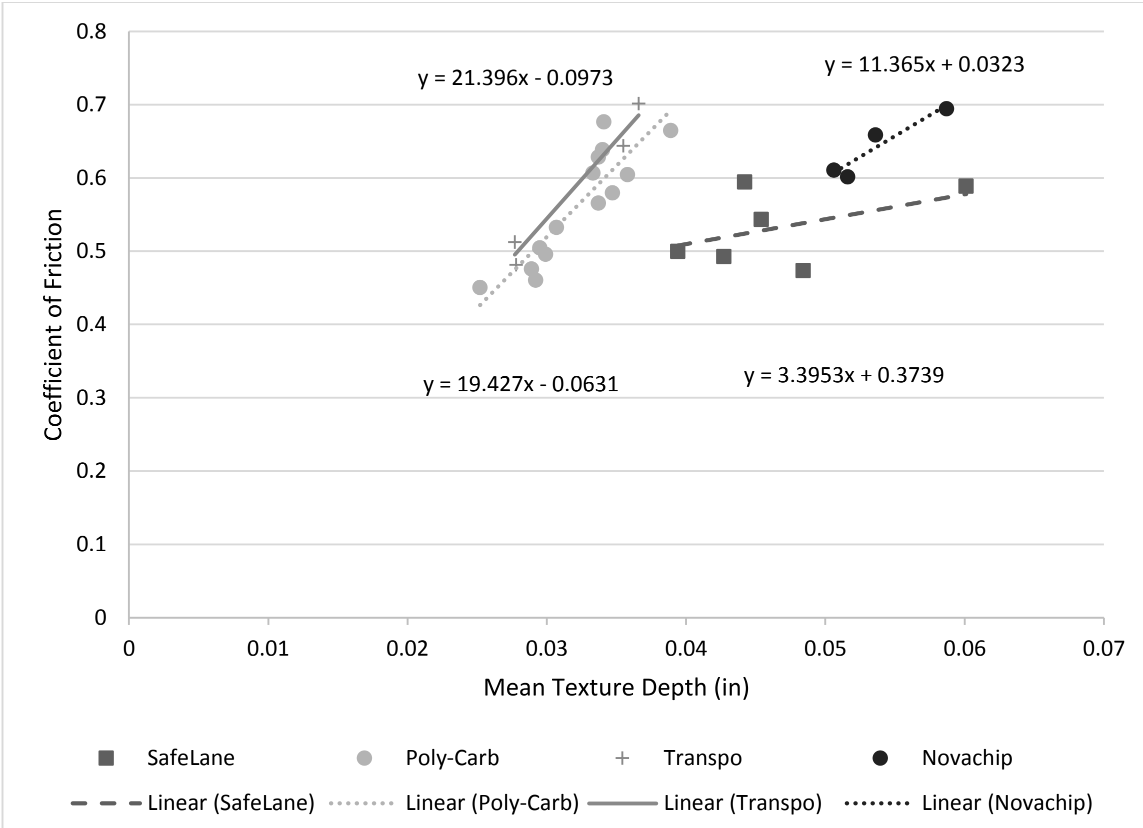


Figure 84: Trends between the Mean Texture Depth and Coefficient of Friction for the Different HFO Systems

The rate at which the coefficient of friction increases can be broken into two different groups. The first group is Poly-Carb and Transpo systems, which both use 0.187 inch [5 mm] aggregates. The second group includes SafeLane and Novachip, which use 0.375 inch [9.5 mm] aggregates. The smaller aggregate group's coefficient of friction increases more rapidly compared to the second group. This suggests that smaller aggregates have a higher ability to create a frictional surface compared to larger aggregates. If aggregates become too large, the majority of frictional properties will come from the aggregate's microtexture, whereas tightly grouped aggregates of a smaller size provide frictional properties based off of their macrotexture. Based on this it is recommended that HFO systems use aggregates under 0.375 inches [9.5 mm] in their system in order to utilize the macrotexture of aggregates for frictional properties.

**Chloride Ion Penetration Results**

The following section provides results of the chloride ion content testing conducted by MnDOT as well as an analysis of the data. Tables containing the chloride content data are provided in the Appendix. The average chloride content for Bridge 9823 and Bridge 69006 are below in Figure 86 through Figure 89. The figures are separated into driving and passing lanes for each bridge. It is important to note that the chloride content lower limit at which corrosion can occur (discussed above) is also shown on the following plots. Data points to the left of this lower limit are in the range of possible corrosion. The lines between data points are dashed because there is not a linear correlation between two different depths. For instance the average chloride content at a depth of three inches below the surface shown on the graph is the average content found in cross section of the cores cut between two and three inches below the surface.

Below in Figure 85 the data from Bridge 9823 in 2011 is displayed. All six cores taken as samples are displayed with a solid line while the averages for each lane are shown in dashed lines. The purpose of this figure is to illustrate the deviation between different samples. Depending on the sample location chloride content can vary upwards of 3000 ppm highlighting one of the downsides of this testing. Although the test method is reliable, the sample size is small and localized relative to the surface of the bridge deck.

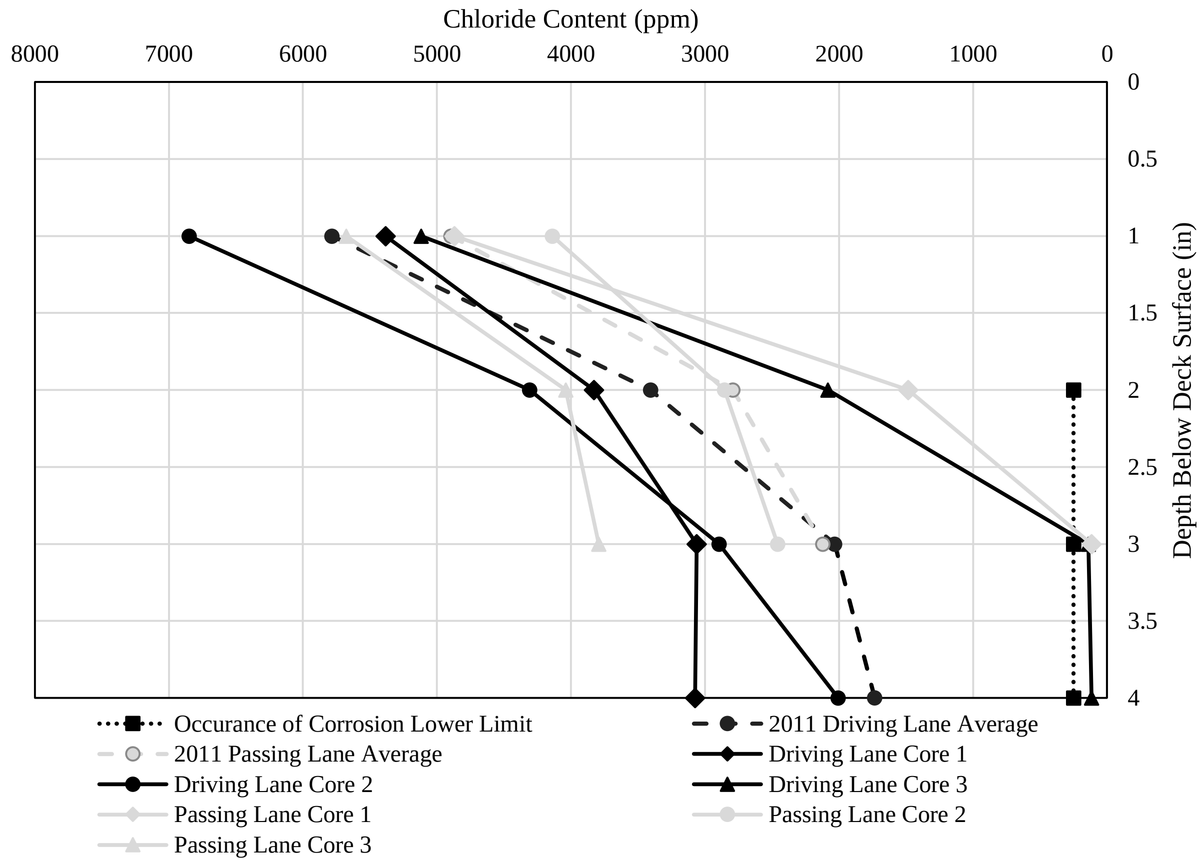


Figure 85: Driving Lane Chloride Content in parts per million during 2011 (Bridge 9823)

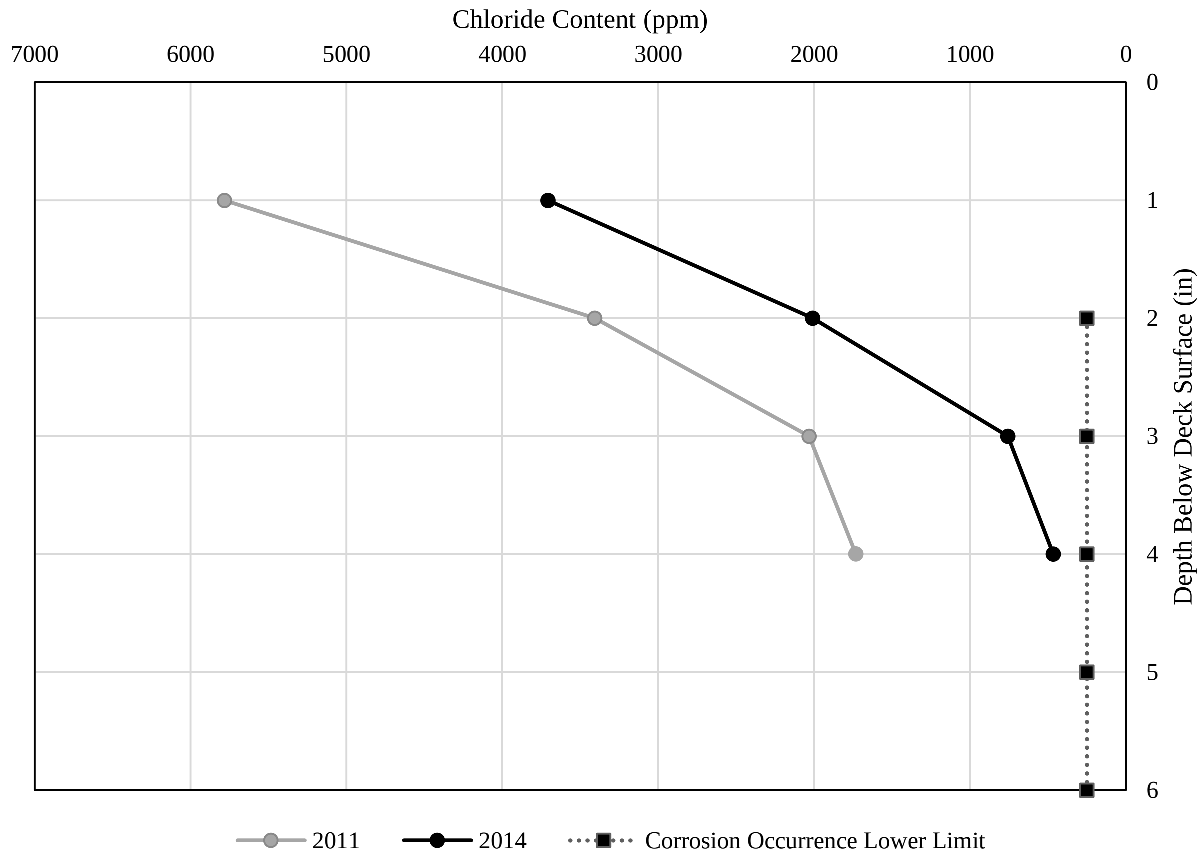


Figure 86: Driving Lane Average Chloride Content in parts per million (Bridge 9823)

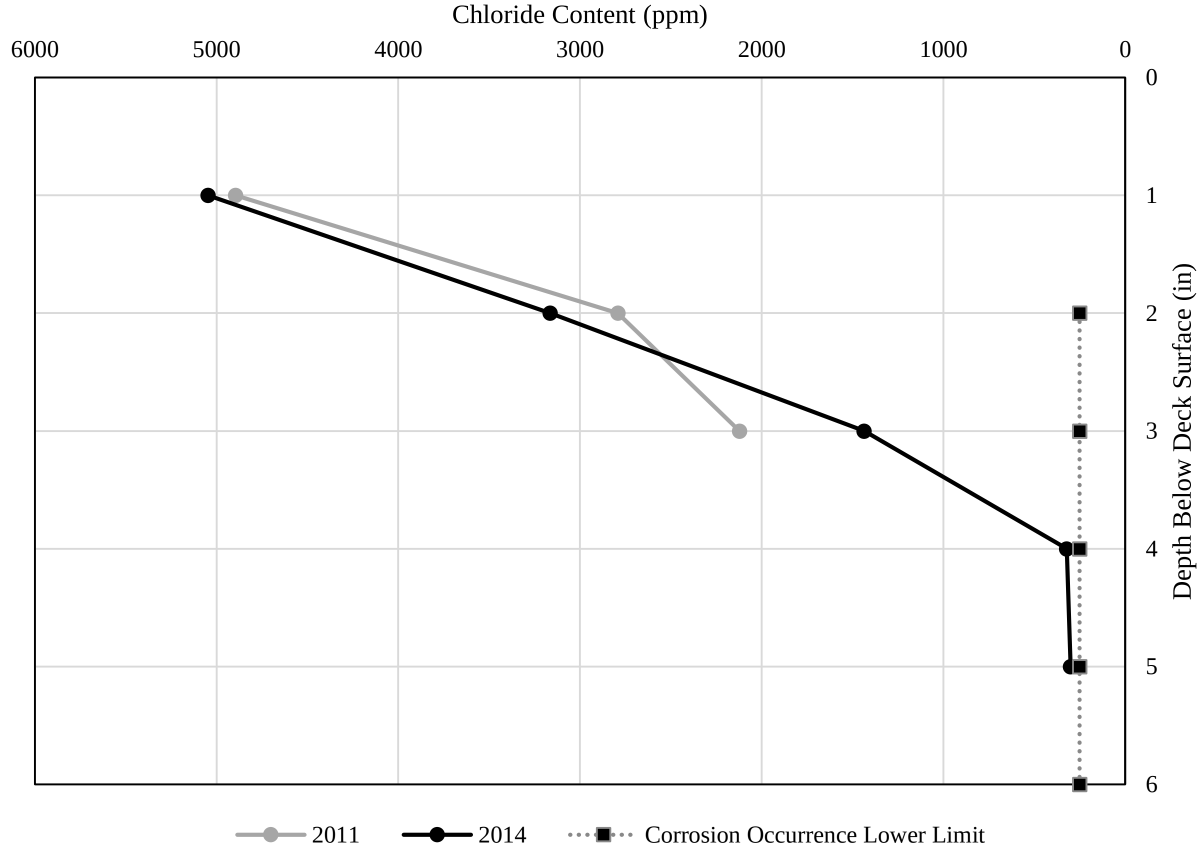


Figure 87: Passing Lane Average Chloride Content in parts per million (Bridge 9823)

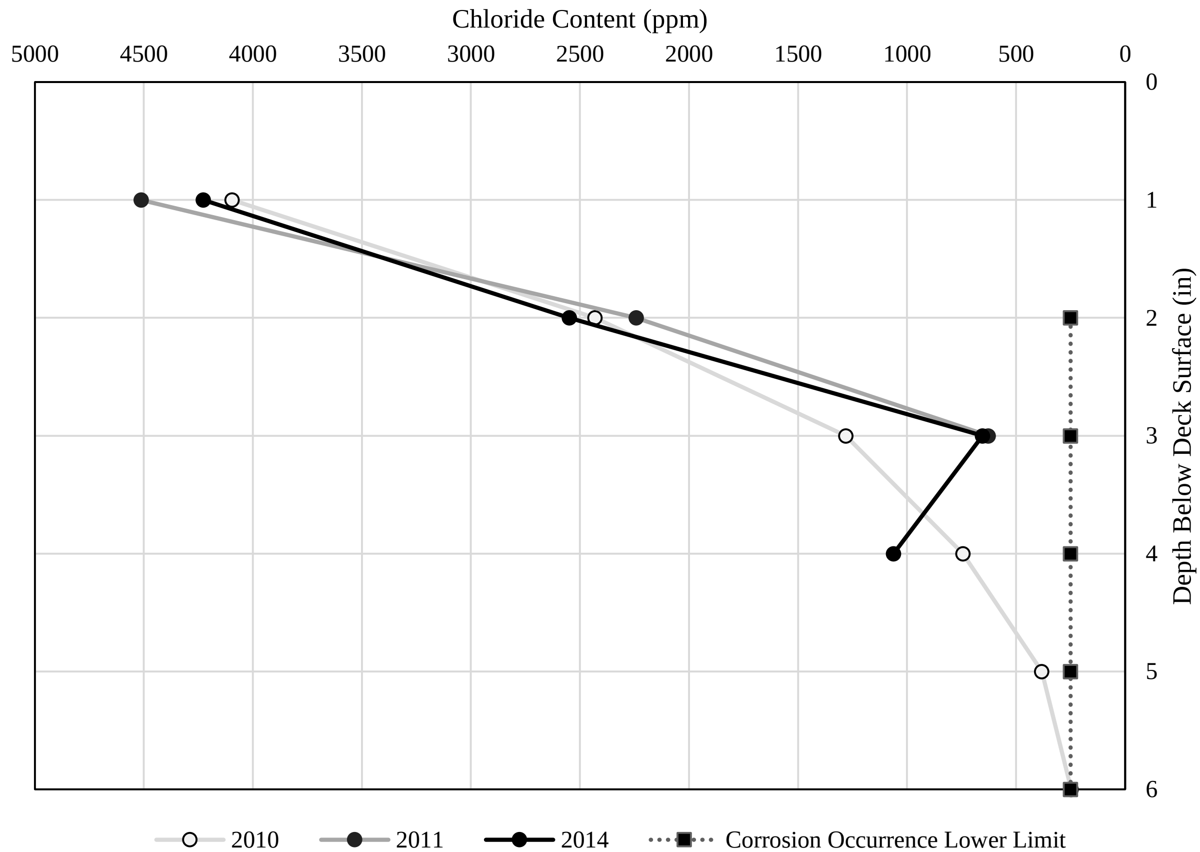


Figure 88: Driving Lane Average Chloride Content in parts per million (Bridge 69006)

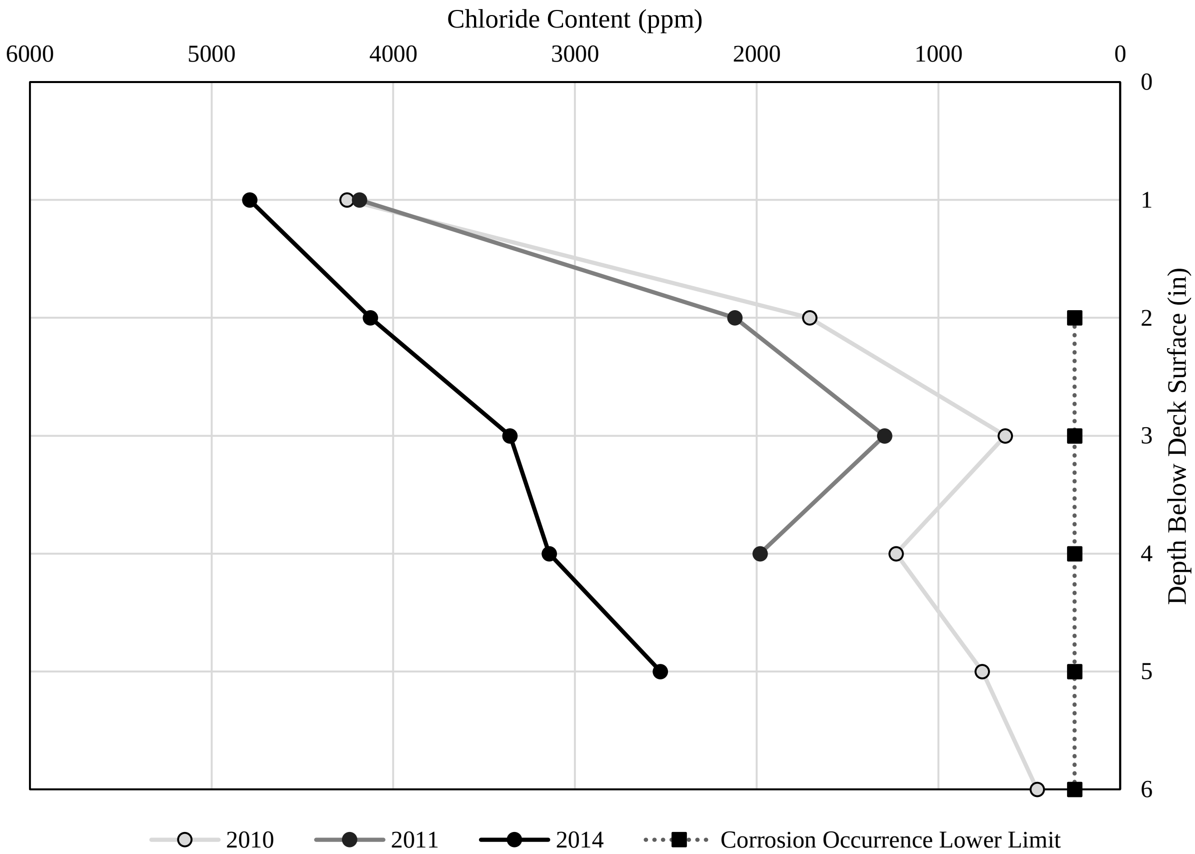


Figure 89: Passing Lane Average Chloride Content in parts per million (Bridge 69006)

In large, the chloride content at similar depths remains relatively constant or decreases throughout the various years of testing. This trend is true for the driving lanes of both bridges as well as the passing lane of Bridge 9823. For example the chloride content for the driving lane of Bridge 69006 at a depth of two inches remains around 2500 ppm, while at three inches it drops from around 1200 ppm in 2010 to near 650 ppm in 2014. It should be noted that all testing was conducted after the HFO systems were installed, however the bridge decks were constructed years before the installation of an HFO system. Therefore it is reasonable to assume that the chloride content found in these samples penetrated the bridge deck before the installation of an HFO system.

This trend does not occur in the passing lane of Bridge 69006 (Figure 89). In this lane the chloride content at a depth of two inches increased from 1700 ppm in 2010 to 4100 ppm in 2014, and this increasing trend occurs along all depths on this lane. The reason for this is that the sample taken in 2014 must have been cored at or near a distress in the HFO system where the system's impermeability properties have been compromised. Distresses that compromise the system's properties include delamination, cracking, or removal of the system due to snow plow abrasion. When compromising distresses occur, they provide an environment that allows for the infiltration of chloride ions into the bridge deck.

The trends described above prove that when properly placed, and when compromising distresses have not occurred, HFO systems have the ability to protect infrastructure from chloride ion penetration. However, distresses of the system such as delamination, abrasion, or cracking, mitigate this protection in a localized area near the distress. It is also important to note that these systems do not help to reduce the amount of chloride ions that had penetrated into the concrete bridge deck prior to the installation of an HFO system.

Based on the field observations conducted, of the fourteen bridges in this study:

- Two bridges had delaminated HFO systems (Br 4190 and 9036, both Poly-Carb)
- Three bridges had cracks in the HFO systems (Br 27019, 86013, and 86019 - all Novachip)
- One bridge had delamination an cracking (Br 6347, Transpo)
- One bridge had cracking and complete removal of system sections due to abrasion (Br 27758, Transpo)

Thus fifty percent of the bridges in this study have compromised systems where water and chloride ions are able to penetrate into the bridge deck. The harsh winter conditions and maintenance practices performed in Minnesota facilitate the distress of HFO systems in such a way that the system's ability to protect bridge decks from chloride ion penetration can be compromised near such distresses.

## Field Observations and Testing Summary

In this task a number of field tests and observations have been conducted to gain perspective on the performance of bridge decks with epoxy system overlays. This task not only evaluated how the systems have been performing, but also provided information regarding changes in year-to-year performance by comparing results from Year-1 to Year-2. The main concern for the wear courses assessed through testing include: (1) retention of aggregate particles, (2) degradation and polishing of aggregate particles due to the actions of traffic loading and snow plow abrasion, (3) frictional characteristics of the wear courses, (4) deck sealing capabilities, and (5) durability of aggregate under repeated freezing and thawing conditions.

The HFO systems visited are performing at varying states of satisfaction. The three distresses: (1) aggregate raveling, (2) abrasion, and (3) polishing are apparent on all bridges. Severe winter conditions and maintenance common in Minnesota appear to cause abrasion that rapidly reduces the life of a high friction overlay system.

Several conclusions can be drawn about the skid numbers and abrasion when results are compared to other pavement types:

- Novachip systems perform better in regards to abrasion. The viscoelastic response of asphalt allows for the continual embedment of aggregates into the matrix which helps to reduce the amount of aggregate loss.
- Novachip systems have a superior aggregate retention which results in an ability to maintain higher friction values.
- Novachip systems have the lowest performance in the torque bond test because of asphalt’s inferior tensile strength compared to epoxy.
- Novachip systems appear to have some permeability into the system. However the large aggregate on the surface prevent an adequate seal for testing equipment. Therefore it is not possible to determine the rate of permeability and how deep water is able to penetrate.
- Poly-Carb systems had the fewest sample removals during torque testing.

- Significant abrasion, especially in the non-wheel path, occurred on the Poly-Carb and SafeLane bridges when comparing their MTD values from 2013 to 2014.
- Dynamic Friction Testing was conducted in order to acquire friction values specific to the wheel and non-wheel path. The coefficient of friction results are comparable with skid numbers, however DFT testing is able to reduce some uncertainties associated with skid testing.
- The two Transpo systems vary heavily from each other. The system in Osceola (Br 6347) is a more complete system (thicker system, larger aggregates, less distresses). The system in Minneapolis (Br 27758) is very worn down with the bridge deck exposed across a significant portion of the bridge.
- The combination of water presence under a bridge and poor expansion joints may play a role in the delamination of epoxy systems from a bridge deck due to moisture presence and expansion/contraction of the bridge deck.
- Correlations between skid numbers (skid trailer) and coefficient of friction (dynamic friction tester) exist. Both tests provide similar results; DFT testing is preferred because of its localized testing ability and its reduction in errors due to testing in a moving vehicle.
- Higher friction values are achieved faster while increasing the mean texture depth (MTD) through the use of smaller aggregates (0.187 NMAS compared to 0.375 NMAS).
- When properly placed, and when compromising distresses have not occurred, HFO systems have the ability to protect infrastructure from chloride ion penetration.
- When distresses such as delamination, cracking, or removal of the HFO system occur, the HFO system's ability to protect the infrastructure from chloride ion penetration can be compromised. Once compromised, chloride ion penetration will occur in local areas of the bridge deck near the distresses. Based on field observations from other parts in this study, fifty percent of the bridges being researched have such compromising distresses in their HFO systems.

# Chapter 5: Crash Data Analysis

This section analyzed crash data to evaluate the effectiveness of high friction overlays in reducing crashes. This study was completed by analyzing ten years of data for nine bridges, encompassing four different proprietary overlay systems. Within one of the overlay systems, three different aggregate types were also compared. Crash characteristics analyzed included the crash time, weather conditions, bridge surface conditions, average daily traffic (ADT), and severity of crashes.

The analysis of data suggests that although there is a decreasing trend in overall crashes, a reduction in crashes on bridges cannot be completely attributed to the use of high friction overlays. The presence of high friction overlays may be unable to play a role in winter crash prevention.

## Causes of Crashes in Minnesota

In order to reduce the number of crashes occurring on a roadway, a transportation agency must first identify what is causing the crashes. The following is a summary of why crashes in Minnesota are occurring. Data was taken from the Minnesota Motor Vehicle Crash Facts 2013 Report. This report is published annually by the Minnesota Department of Public Safety. About one-third of all crashes are single vehicle accidents. For single-vehicle accidents, illegal or unsafe speed is the most common cause followed by inattentive or distracted driving. In the multi-vehicle accidents that account for the other two-thirds of total accidents, inattention or distraction is the most common cause (MN Department of Safety, 2014).

In 2013, 77,707 crashes were reported to Public Safety in Minnesota. This was an increase of 10.9% from 2012. Deaths were reduced by 2.0% from 2012 with 387 occurring in 2013. It should be noted that 2013 displayed harsh winter conditions above the state's average. Although not stated by MnDOT or any other organizations, this study attributes the harsh winter conditions for the increase in crash rates between 2012 and 2013. The number of crashes occurring in 2013 was the highest number of annual crashes to occur in Minnesota since 2008. When looking at longer trends of crash rates in Minnesota, the number of crashes occurring each year generally decrease. The number of crashes occurring in the state is down from 94,048 in 2003 to 69,236 in 2012 (MnDOT, 2013).

This decreasing trend is occurring both in overall crashes as well as critical emphasis areas identified by state agencies. Critical emphasis areas are crash specific characteristics that are used to help identify specific ways to reduce crashes. These areas are based on guidelines set by the Strategic Highway Safety Plan (SHSP) created by the Federal Highway Association (FHWA). The SHSP recommends that states should identify between four and eight emphasis areas to help direct efforts and make a more "strategic" and effective plan for improving safety (FHWA, 2013). The Minnesota emphasis areas listed in

Table 21 and were identified by state agencies based on an analysis of safety data (MnDOT, 2014B). This decreasing trend provides proof that the increase in education, engineering, enforcement, and emergency medical and trauma services through the Towards Zero Deaths program is helping to provide safer driving conditions.

Table 21: Percent Change of Crash Emphasis Areas

| Emphasis Area | Percent Change 2003 to 2012 | Percent Change 2008 to 2012 |
|---|---|---|
| All Fatal & Serious Injuries | -51% | -17% |
| Unbelted | -66% | -30% |
| Impaired | -41% | -13% |
| Speed | -67% | -35% |
| Inattention | -64% | -24% |
| Lane Departure | -46% | -16% |
| Intersection | -57% | -21% |
| Older Driver | -39% | -90% |
| Young Driver | -69% | -38% |
| Commercial Vehicles | -42% | -14% |

According to the National Highway Traffic Safety Administration, the causation of a crash is attributed to the driver 94% of the time, the other 6% is evenly distributed between the vehicles, environment, and other unknown reasons (USDOT, 2015). While the vast majority of crashes occur due to human error, the elimination of poor roadway conditions through engineering and proper maintenance can assist in a reduction of crashes. Since HFO systems are designed to create higher friction surfaces that help to reduce crash rates, this study focuses on crash characteristics pertaining to the surface conditions.

## Data Collection

In total, nine bridges in six different locations were included in the crash reduction study. The two bridges excluded from the crash data analysis were Bridges 27758 and 86019. Bridge 27758 (Penn Ave, Poly-Carb) was excluded because only one accident was reported in this section during the ten years of provided data. The event occurred after the epoxy overlay installation, however it occurred with clear and dry conditions during rush hour and therefore it can be assumed this accident was caused from extraneous factors that the overlay surface could not prevent.

Bridge 86019 [Novachip] is a southbound ramp that connects traffic from CASH 36 to TH 101 in Otsego, MN. The bridge crosses over Crow River. During the ten years of data provided there were no reported accidents.

The following comparison of crashes and certain site conditions between the nine bridges used data from the Public Safety Database provided by MnDOT. The study compared eleven years of data from 2003 through 2013. This data provides information from both before and after installation of the HFO systems. The installation dates for these systems vary between 2007 and 2010. When the data was procured for bridges in the Metro district (Bridges 6347, 27019, and 86013) the data from 2003 was incomplete and was therefore omitted from the analysis. For the same three bridges, the data for 2013 extends through May while the six bridges from other districts have data through December of 2013. Finally it should also be noted that Bridge 6347 crosses the St. Croix River connecting Minnesota and Wisconsin. For this study only data for the Minnesota half of the bridge was provided and analyzed.

## Analytical Comparison

In this analysis a total of 293 crashes were included for the nine bridges. The public safety database provides an abundance of information pertaining to each crash however not every factor provided a useful comparison for this report. Several aspects of the crashes were investigated, including:

- Total number of crashes
- Bridge deck surface condition
- Weather
- Before and after HFO system installation
- Average Daily Traffic (ADT)

It is important to note that for some of the bridges in this study, the total number of crashes is very small; analysis for some of these bridges may appear skewed due to the small sample size. This skewing must be noted when comparing different statistics in percentages, and is especially true for Bridge 86013 [Novachip] and Bridge 6347 [Transpo] where only two and four crashes occurred respectively during the ten year time span.

**Cause of Crashes**

Of the 293 crashes occurring on the nine bridges, 40 did not list a contributing factor. For the remaining 253 crashes the following are the top five contributing factors listed with their percentage of occurrences.

1. Illegal or unsafe speed, 37%
2. No clear factor, 23%
3. Following too closely, 8%
4. Inattentive/Distracted Driving, 8%
5. Weather, 7%

A second factor was listed in about half of the crashes with the top four secondary factors including: weather (30%), inattentive/distracted driving (15%), skidding (11%), and illegal or unsafe speed (10%). While a majority of accidents are attributed to human error, weather and surface conditions of the bridges exacerbate conditions in which speeding or distracted driving may result in a crash. This data reinforces the focus on crash characteristics pertaining to the surface conditions for bridges with an HFO system installed.

Another factor that may lead to crashes occurring includes the physical factors of a roadway, including geometry, elevation, and traffic levels on the bridge. Geometry and elevation of the bridges were not covered in the scope of this research. A strong correlation exists when comparing the total number of crashes occurring to the Average Daily Traffic (ADT). As shown below in Figure 90, as the ADT increases, so does the number of crashes. This is a fairly intuitive result, however it helps to illustrate that many factors other than the friction values of the bridge surface cause a crash. As ADT increases, the proximity of vehicles increases, and driving tendencies may change with other vehicles present on the bridge.

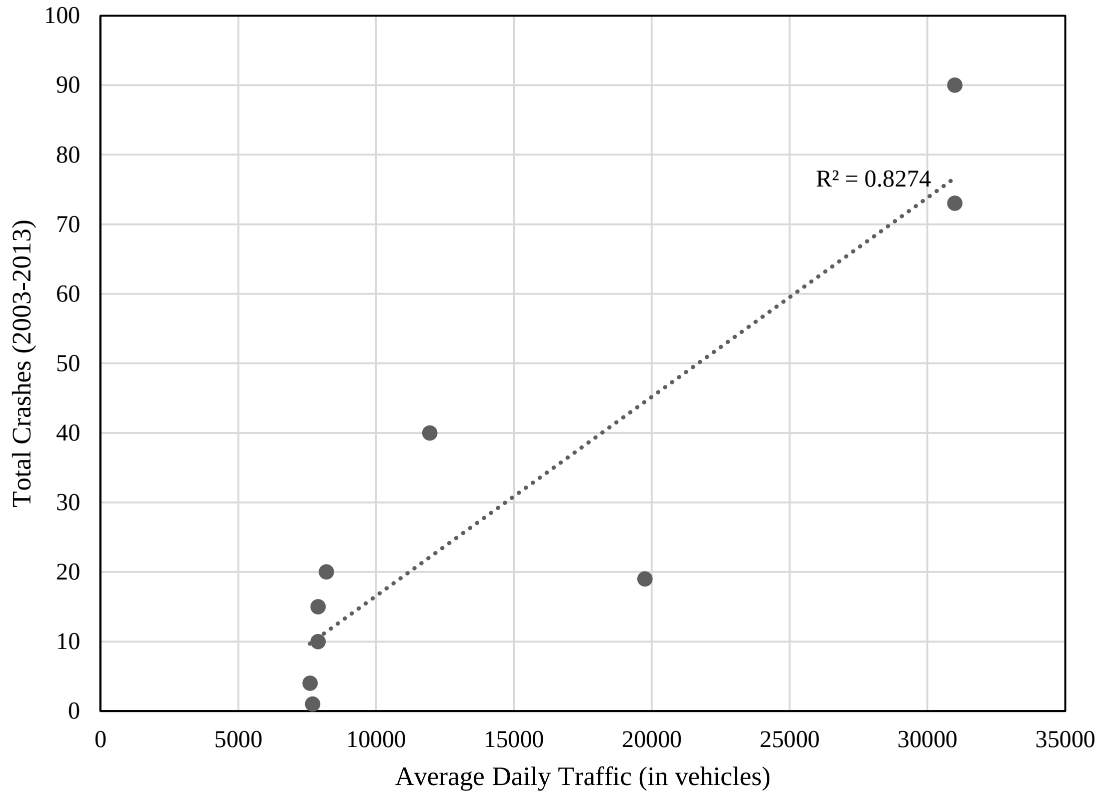


Figure 90: Total Crashes Compared to the ADT

**Severity of Crashes**

The Public Safety Database has the severity of a crash broken down into five different categories. These categories are based on bodily injury caused by the crash ranging from no apparent injury to fatality. The majority of crashes (76%) did not result in any injury. Two separate crashes occurred on westbound I-94 in Barnesville (Bridge 14809 [SafeLane]) that resulted in a fatality. A fatal crash occurred both before and after the HFO system was installed.

**Time of Crashes**

A majority (75%) of the crashes occurred during the winter months (October-March). This is expected because of poor driving conditions and roadway surfaces due to inclement weather and the presence of water, snow, and ice on roadways. HFO systems were designed to improve roadway conditions during this time of poor weather. The main way these systems were designed to improve roadway conditions is through their coarse macrotexture created from aggregates with a high microtexture. Both microtexture and macrotexture are important to improving friction values.

**Surface Conditions**

The surface conditions that were present during crashes are provided in Table 22. Dry and ice packed snow conditions were by far the two most common surface conditions at 37% and 39% respectively. It should be noted that a large percentage of crashes occur on a dry surface due to the fact that roadways are dry for a majority of days.

Table 22: Surface Conditions during all Crashes

| Surface Condition | Percent of Crashes (%) |
|---|---|
| Dry | 37 |
| Ice Packed Snow | 39 |
| Slush | 3 |
| Snow | 9 |
| Wet | 7 |
| Other | 5 |

Ice packed snow conditions are at the opposite end of the spectrum compared to dry conditions. While ice and snow produces slippery conditions on roadways, it also reduces the effectiveness of the macrotexture found in HFO systems. As snow falls onto the bridge surface snow and ice are packed into the macrotexture by traffic loads crossing the bridge. The snow and ice fill the voids between individual aggregate particles reducing the friction available from a system (Figure 91).

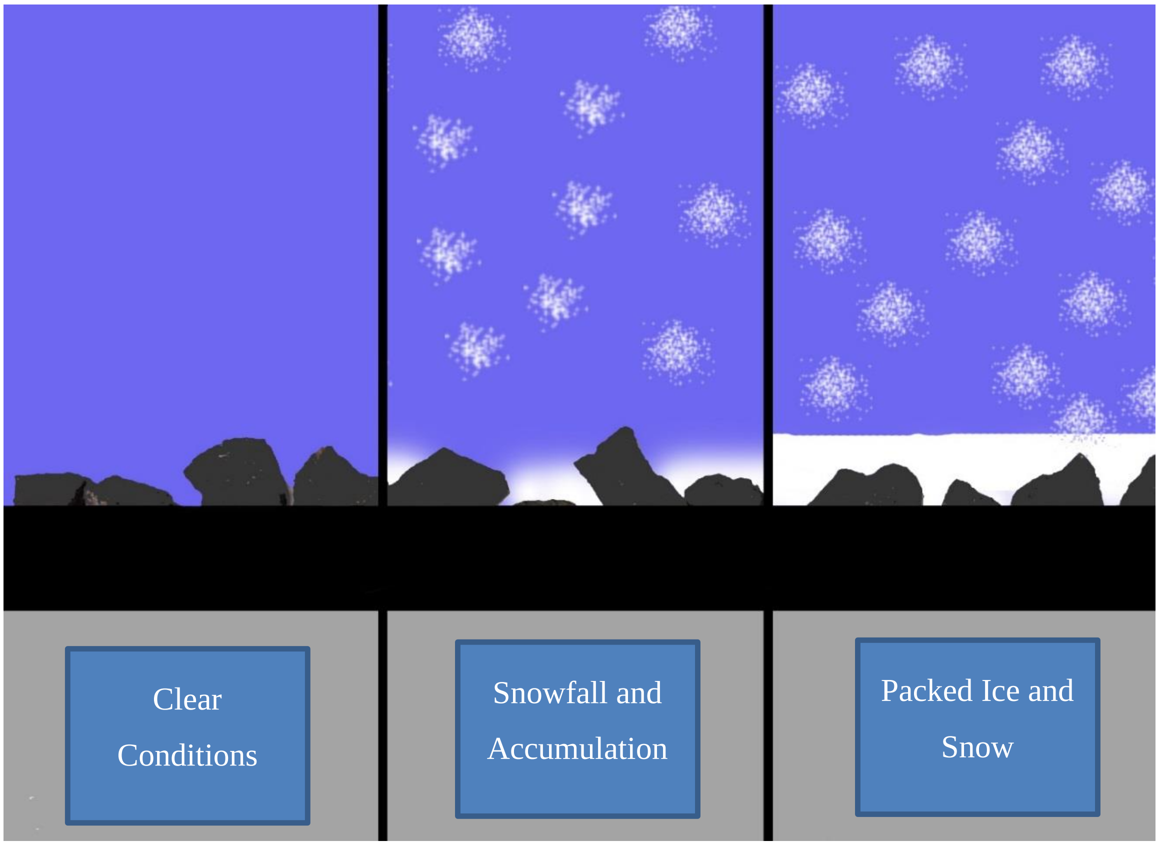


Figure 91: Accumulation of Packed Ice and Snow due to Traffic Loads.

With the assumption that HFO systems have a greatly reduced effectiveness during ice packed snow conditions, and their purpose is not to prevent crashes during dry conditions, it can be concluded that 76% of the crashes occurred with surface conditions during which high friction systems are covered and therefore are not effective.

A total of 55% of these crashes occurred when the weather was either clear or cloudy. This means that half of the crashes with this surface condition occurred during a time when inclement weather was not present. Such a high percentage presents the conclusion that many of the crashes occurred directly as a result of the surface containing ice packed snow. Thus the removal of ice packed snow surface conditions would help to reduce the number of crashes on bridge decks. Several methods currently exist to reduce this condition including automated deicing systems.

The two bridges in Moorhead, Bridges 9066 and 9067 [Poly-Carb, Flint], have an automated deicing system in place. Such a system uses sensors embedded in the bridge deck to detect potential freezing conditions. When these conditions occur the deicing chemicals are sprayed onto the bridge deck using an embedded application system. The main purpose of such a system is to prevent ice buildup on bridge decks. This is different than traditional deicing techniques such as salt which are used to break down ice that has already formed on the surface. Even with a deicing system in place, 34 of the 73 (47%) crashes on Bridge 9066 and 15 of the 90 (17%) of the crashes on Bridge 9067 occurred with ice packed snow surface conditions. For this report there is no information regarding these deicing systems effectively preventing any conclusions from being drawn. Several factors may account for ice packed snow conditions occurring even with these systems present including (1) when the system was installed, (2) performance of systems, (3) maintenance required, and (4) if the system was not functioning during winter months. Looking further into crashes with ice packed snow surface conditions and better methods to effectively remove such conditions could help to further reduce the number of crashes occurring on bridges in cold climates.

## Crashes Before and After HFO System Installation

One of the most important aspects to analyze performance of HFO systems is the crash rates before and after the installation of such a system. The following provides such a comparison. In general the average number of crashes annually decreases after installation. The average number of crashes on the bridges was reduced from 3.52 crashes per year before an HFO system to 2.60 crashes per year. In other words an annual reduction of nearly one crash per bridge per year. Several factors including severity of crashes and surface condition were analyzed to verify whether systems with a higher friction bridge surface provide a safer bridge deck.

**Severity: Before and After**

The comparison of crash severity is broken down into the five different categories based on bodily injury. Overall the distribution of injury did not change with the installation of an HFO system (Figure 92). The number of crashes without injury was the largest category to increase at 5.4% from 71.2% to 76.6%. It does not appear that HFO systems play a role in decreasing the severity of a crash that occurs on such a surface.

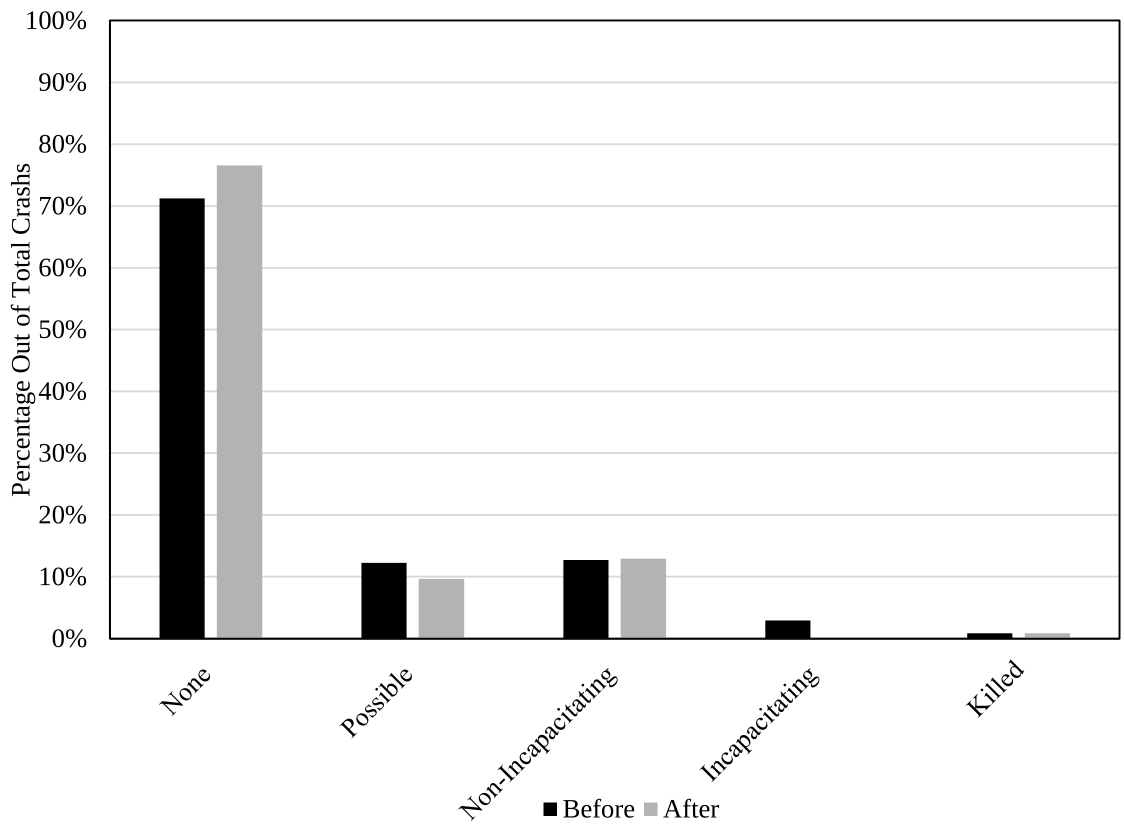


Figure 92: Severity of crashes before and after installation of an HFO system.

**Surface Conditions: Before and After**

The following provides a comparison of the surface conditions when crashes occurred both before and after installation (Figure 93). The solid bars on the plot represent crashes before installation while the bars with cross hatching are crashes after the installation of overlays. It should be noted that the "combined" bars are a conglomerate of wet, slush, and snow surface conditions. In general the annual number of crashes that occurred during the “combined” and “ice packed snow” conditions decreased. On average the “combined” crashes were reduced by 0.3 crashes per year. The “ice packed snow” condition crashes were reduced on average by 0.8 crashes per year. The average number of crashes per year on dry surfaces remained constant for most bridges. Before installation the average number of crashes on Bridge 9067 (Poly-Carb, Flint) was 3.6 crashes per year and doubled to 7.4 crashes per year after the installation of the HFO system. There are not any factors about the crashes or bridge that provide information as to why this rate increased.

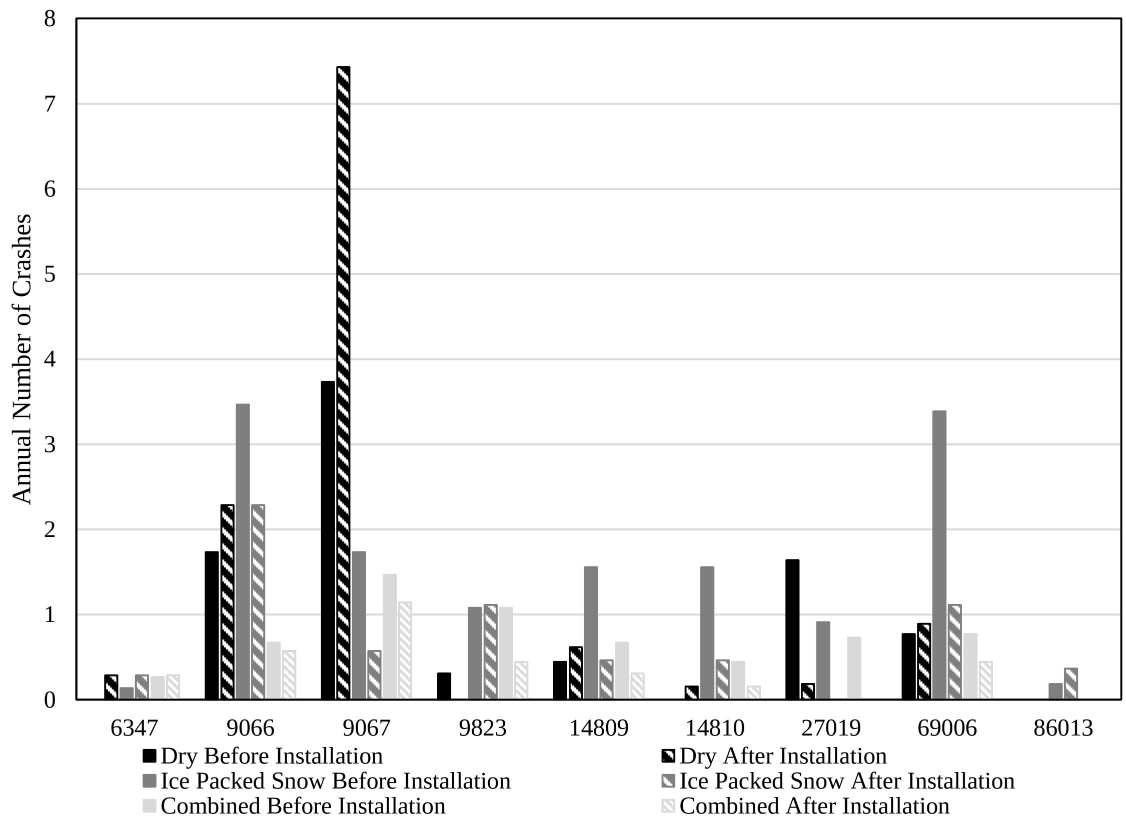


Figure 93: Average number of crashes annually per surface condition before and after installation of HFO systems.

**Normalized to Traffic Levels: Before and After**

The traffic levels of a roadway can also play a large role in the amount of crashes that occur. The average number of crashes that occurred was normalized to the average daily traffic (ADT). The general trend is once the HFO system was installed the number of crashes per million vehicles was reduced. However the amount at which they were reduced is not that significant. On average after a system was installed the crash rate dropped by 0.3 crashes per million vehicles crossing. This translates to a reduction of about 1 or 2 vehicles per year. The bridge with the best performance was Bridge 27019 [Novachip] with a reduction of 5.8 crashes per year. There were eighteen crashes that occurred during the five years before installation of an HFO system. In the five years after installation there was only one recorded crash. This bridge shows the largest reduction of crashes analyzed in this study.

## Comparison of Year-1 and Year-2 Crashes

The following is a comparison of the number of crashes that occurred during the first and second year of this study. The years 2012 (before the study), 2013 (Year 1), and 2014 (Year 2) are compared below in Figure 94. The total number of crashes to occur on the nine bridges analyzed in this comparison are tallied below in Table 23.

Table 23: Total Number of Crashes to Occur (2012- Oct 2014)

| Year | Total Number of Crashes |
|---|---|
| 2012 | 13 |
| 2013 | 37 |
| 2014 | 14 |

The number of crashes that occurred in the first year of this study (2013) was nearly triple the number of crashes to occur during the previous year. Through the first ten months of year two more crashes had occurred than in 2012. This number is expected to increase as November and December are two of the top three months in regard to this study's crash rates. It should be noted that Minnesota experienced an abnormally harsh winter season from during the 2013-2014 winter season and this plays an important role in the increase of crashes during this study. From October 2013 to March 2014, 30 of the 51 crashes in this two year study occurred. Thus 59% of the crashes occurred during a 27% span of the total study time.

The crashes that occurred on each bridge are broken down in the summer (April to September) and winter (October to March) months (Figure 94). In the first year of this study every bridge had a winter-time crash, and number of crashes to occur in the winter were the either the same or higher than in the year 2012.
There was also a total of nine summer time crashes in 2013 compared to five that occurred in 2012. Through the first ten months of 2014 three summer crashes have occurred.

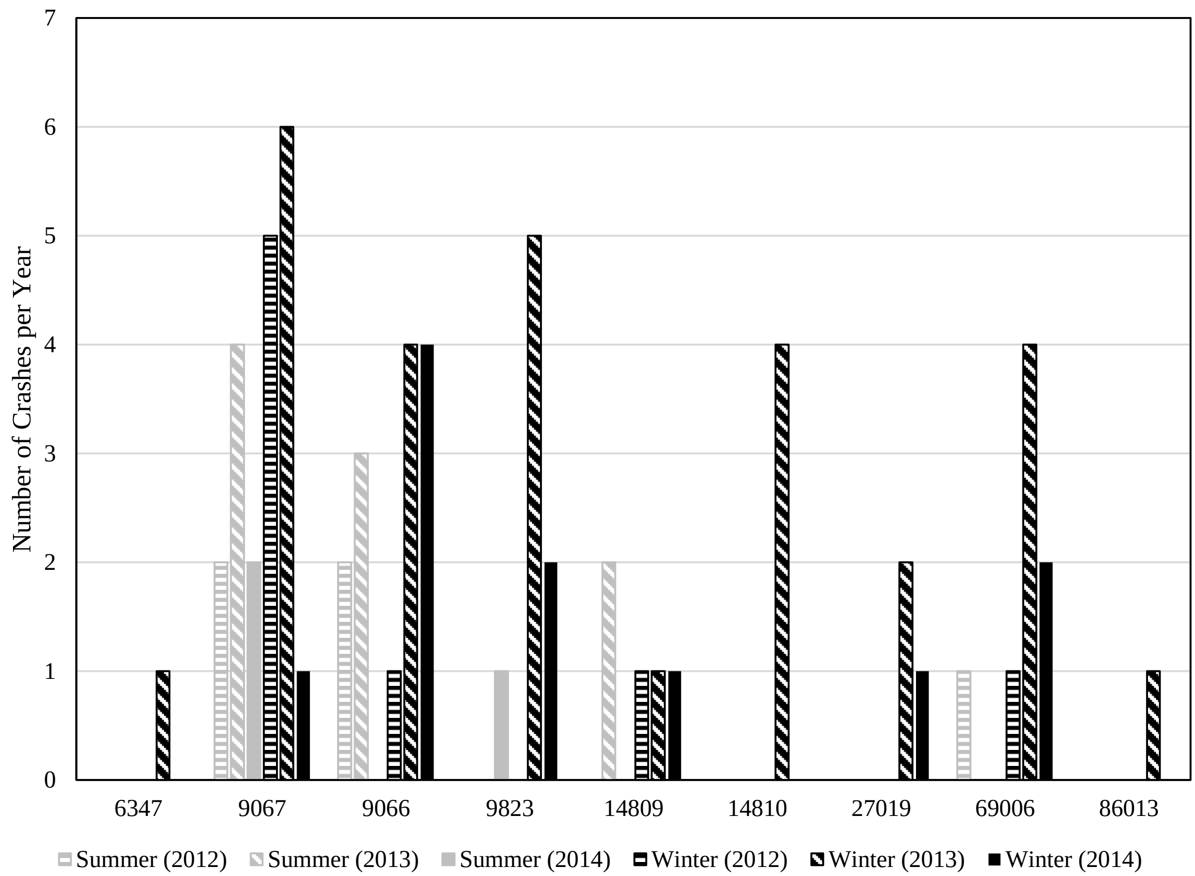


Figure 94: Number of Crashes on Each Bridge during Summer (April-September) and Winter (October-March) Months

The harsh winter season from 2013 to 2014 is also shown with the large amount of ice packed snow conditions present during crashes in 2013 and 2014. Forty-nine percent of the crashes that occurred during 2013 and 62% of the crashes in 2014 had ice packed snow surface conditions present (Figure 95). Ice packed snow surface conditions mitigate the effectiveness of the high friction values found on HFO systems.

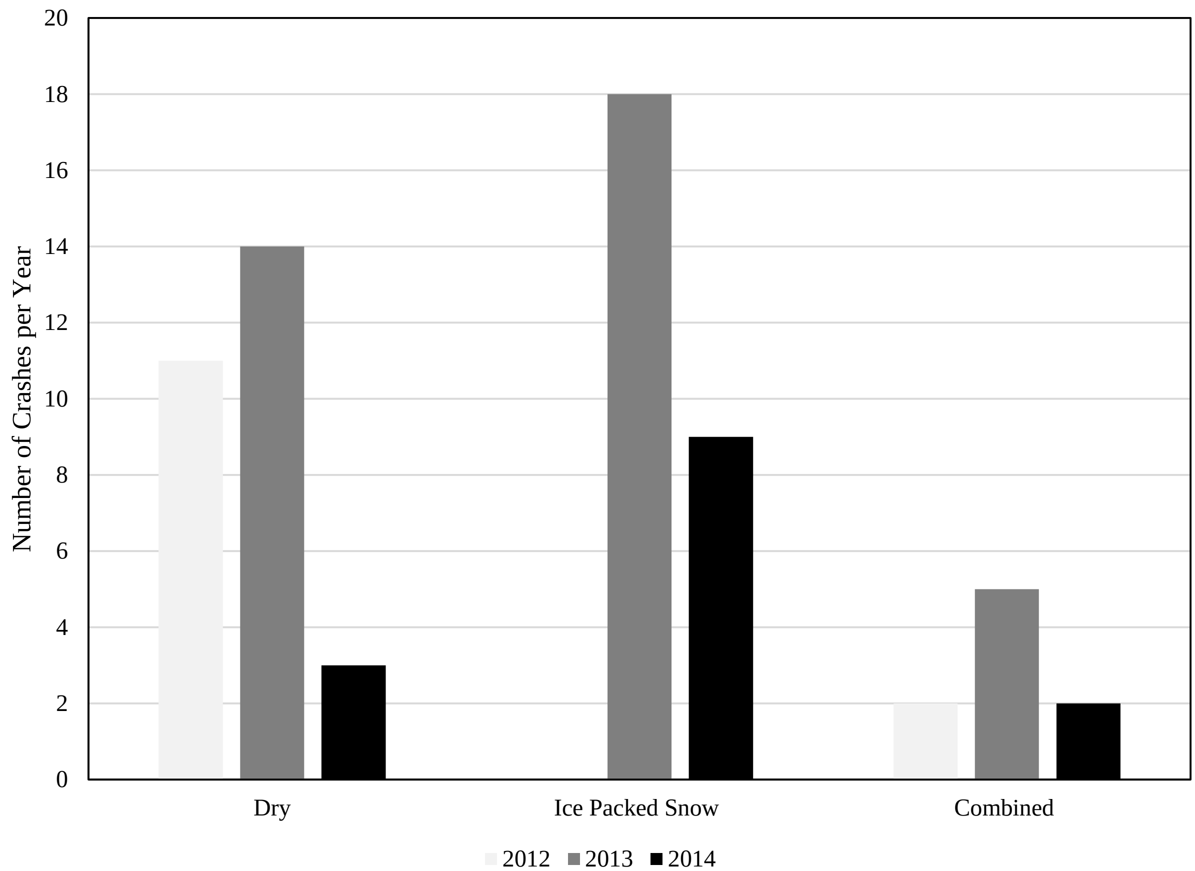


Figure 95: Surface Conditions Present during Crashes in from 2012 to 2014

The comparison of crashes that occurred during the two years of this study reinforce the need to reduce the ice packed snow condition from bridge decks. An HFO system is not an effective way of removing such a condition reducing the value of such a system in Minnesota and cold climate regions.

## Comparison of Different HFO Systems

As discussed previously in the Scope of Analysis section, there are several differences between every bridge in this study. These variables can lead to certain systems to comparatively perform better than other systems. The main variables that may cause differences in performance are the types of systems and the types of aggregates thus a comparison between the four HFO systems may detect whether any particular system is performing better than the others. A similar comparison of the four Poly-Carb bridges is also made in order to compare the influence of different types of aggregates.

### Comparing the Four Different Proprietary HFO Systems

In order to compare the different types of systems the crash data was averaged from the bridges for each system listed below:

- Poly-Carb (Bridges 9066, 9067, 9823, and 69006)
- SafeLane (Bridges 14809, and 14810)
- Transpo (Bridge 6347)
- Novachip (Bridges 27019, and 86013)

The four different systems were compared using the present surface conditions during crashes (Figure 96). Poly-Carb, SafeLane, and Novachip systems showed a significant decrease in crashes during “ice packed snow” and “combined” conditions. With crash reductions ranging from 35% to 100% for the different combinations of systems and surface conditions. A similar amount of crashes occurred both before and after installation of a Transpo system.

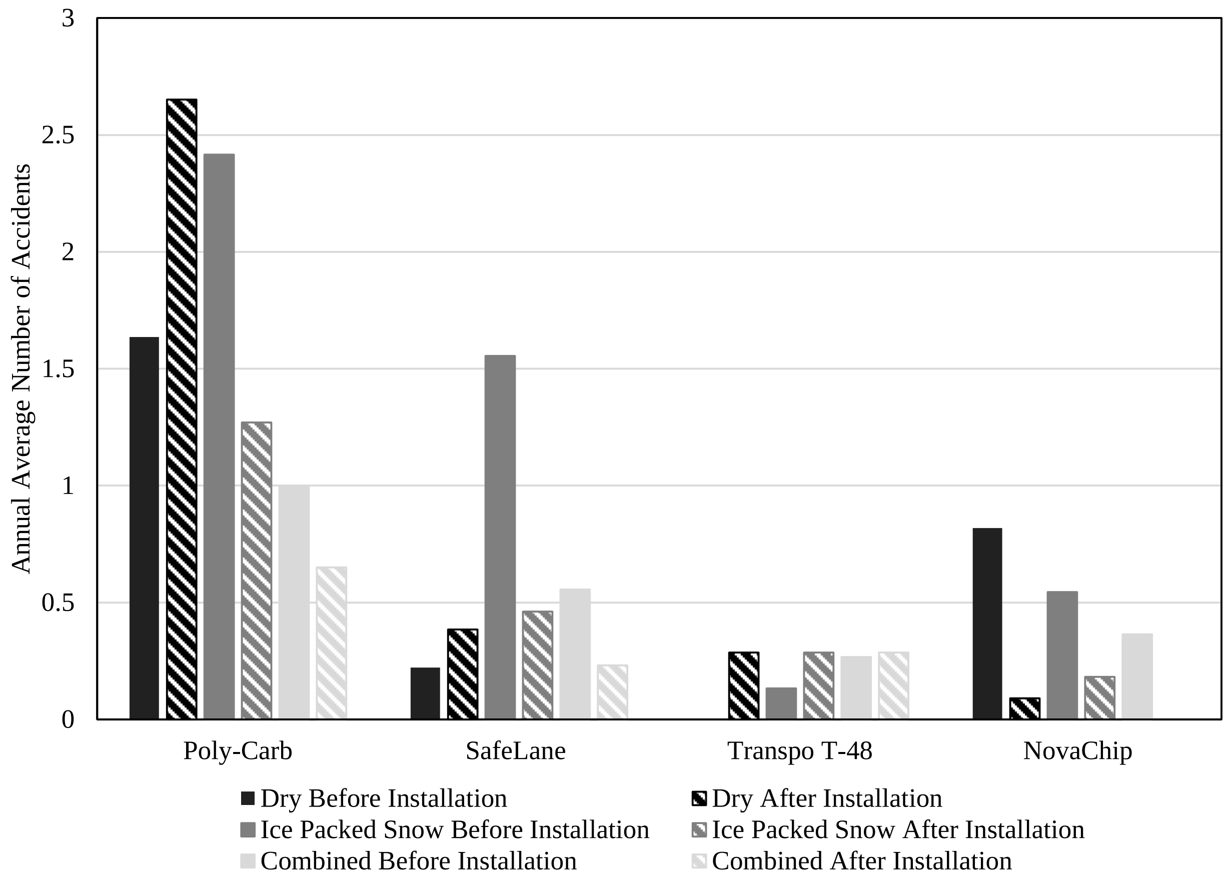


Figure 96: Comparing the average number of crashes annually per surface conditions before and after installation of different HFO systems.

Comparing the amount of crashes occurring before and after installation of the different systems normalized to traffic crossing the bridges reveals similar conclusions as previously mentioned in the section Normalized to Traffic Loads: Before and After. An average reduction of 0.3 crashes per million vehicles which translates into 1 or 2 vehicles per bridge per year occurs after the installation of a system. This is shown graphically below in Figure 97. SafeLane and Novachip systems have the largest reductions normalized to traffic (53% and 88% respectively.) Poly-Carb had a reduction of 27% while the average number of crashes on the Transpo systems doubled after installation.

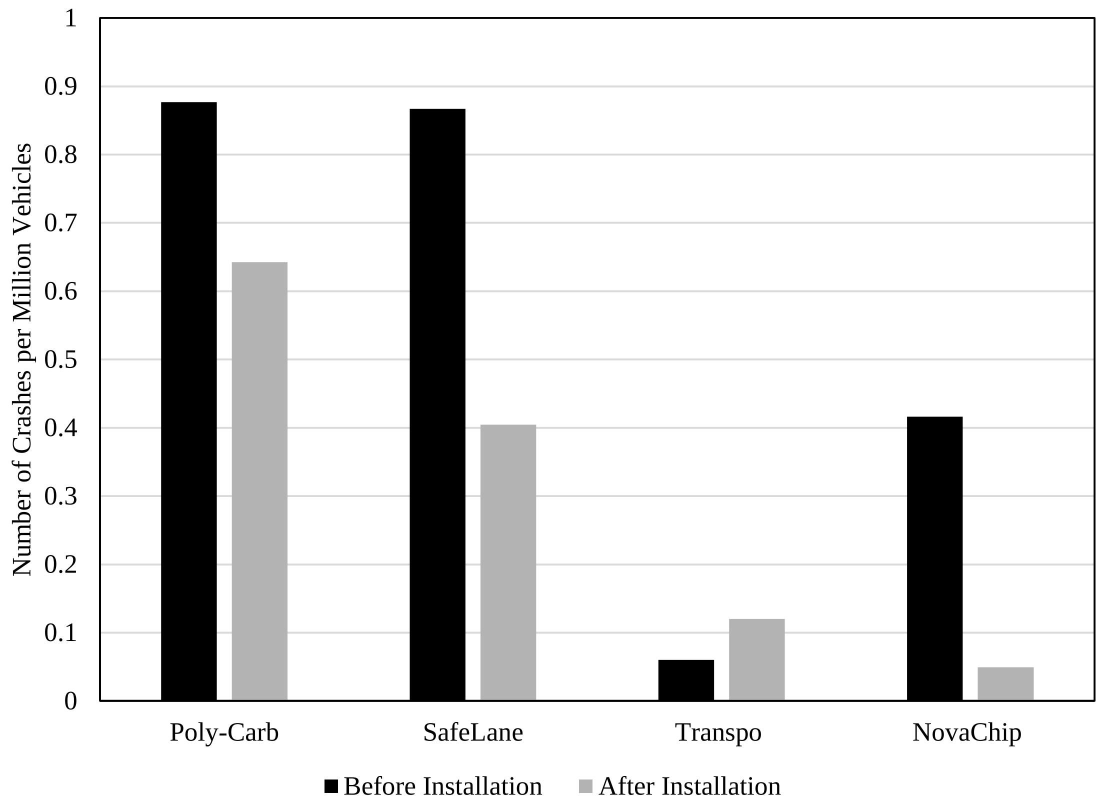


Figure 97: Number of crashes per million vehicles crossing bridge before and after installation of different HFO systems.

Based on the comparisons made in Figure 96 and Figure 97 the Novachip systems appear to have the best performance followed closely by SafeLane and Poly-Carb systems.

**Comparison of Poly-Carb Systems Using Different Aggregates**

The two comparisons discussed in the previous section were also conducted for the different aggregates used within Poly-Carb systems. Three different aggregates are used by the systems in this study:

- Taconite on Bridge 69006 in Virginia, MN
- Flint on Bridge 9066 and 9067 in Moorhead, MN
- Basalt on Bridge 9823 in Atkinson, MN

A comparison of crashes occurring during certain surface conditions before and after installation is shown below in Figure 98. The two most significant themes are (1) the taconite system had a large decrease in the amount of crashes occurring with ice packed snow surface conditions and (2) the number of crashes with dry surface conditions doubled for flint systems.

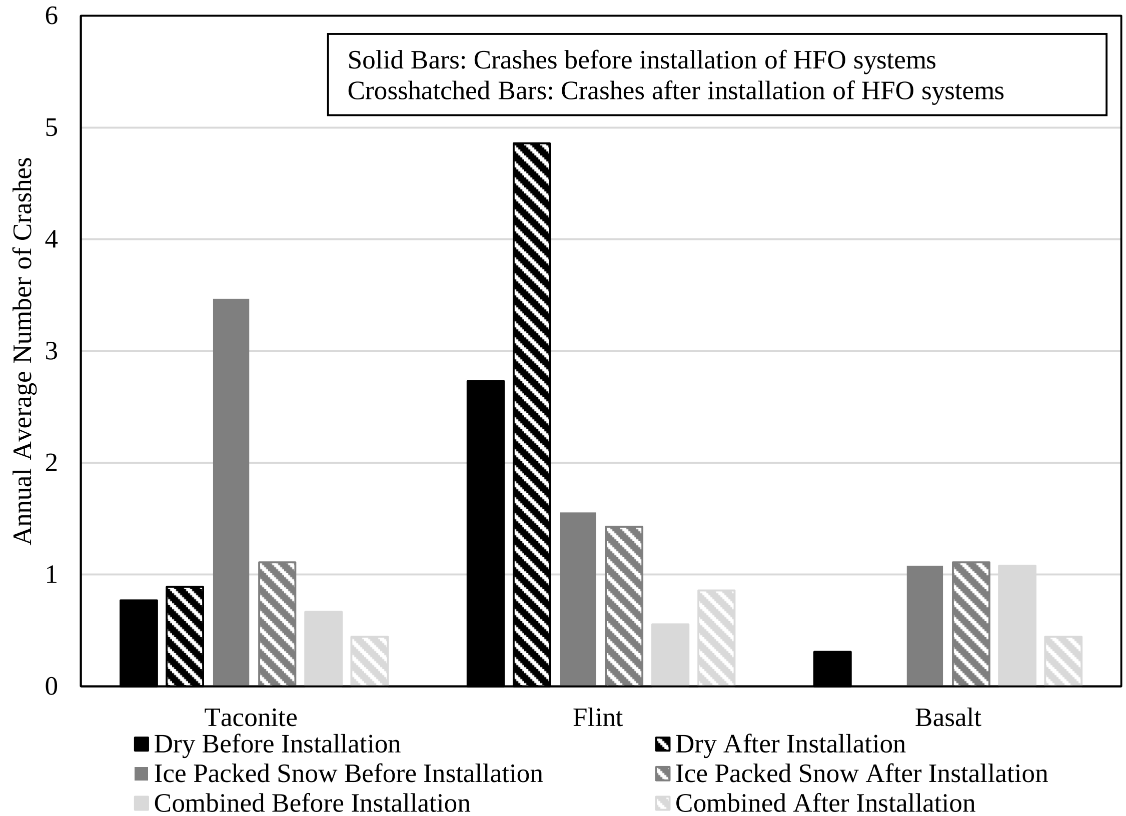


Figure 98: Comparing the average number of crashes annually per surface condition before and after installation of Poly-Carb systems with different aggregates.

When comparing the Poly-Carb systems to one another normalized to traffic, the number of crashes per million vehicles is reduced by 47% and 38% for Taconite and Basalt respectively. However there is not a significant reduction in crashes with the installation of a system using flint aggregates (2%). Based on the comparisons made in Figure 98 it appears that taconite is the best performing aggregate in terms of crashes of the Poly-Carb systems.

**Comparison of Side by Side Bridges with and without HFO System Overlays**

Several locations in this study allow side by side comparison of bridge decks with and without HFO systems. Three different bridges in the study carry one direction of traffic with an HFO system while a counterpart bridge carries the other direction of traffic without. Details for these bridges are provided in Table 24.

Table 24: Details of Bridges in Side by Side Comparison.

| Location | Bridge with HFO system | Bridge without HFO System | HFO Type | Aggregate Type |
|---|---|---|---|---|
| Atkinson | 9823 | 9824 | Poly-Carb | Basalt |
| Otsego | 27019 | 27020 | Novachip | N/A |
| Virginia | 69006 | 69005 | Poly-Carb | Taconite |

In Figure 99 the bridges without HFO systems are denoted with bars that have horizontal hatch patterns. The "after installation" refers to the installation of a single system for each location in order to compare a bridge with and without a system. For example Bridge 69006 had an HFO system installed in 2009, thus the "after installation" for both Bridge 69005 and 69006 is from 2010 through 2013 despite a system not actually being installed on Bridge 69005.

Previous research has looked at crashes occurring before and after the installation of an HFO system. The results provided the perceived benefit of crash reduction, however this perceived benefit occurs on bridges without an HFO installation. As shown in Figure 99, the average number of crashes occurring on a bridge deck is reduced regardless of whether or not an HFO system was installed, and at about the same rate for each location. While crash rates are decreasing on bridges with an HFO system, this reduction cannot be solely attributed to the installation of such a system.

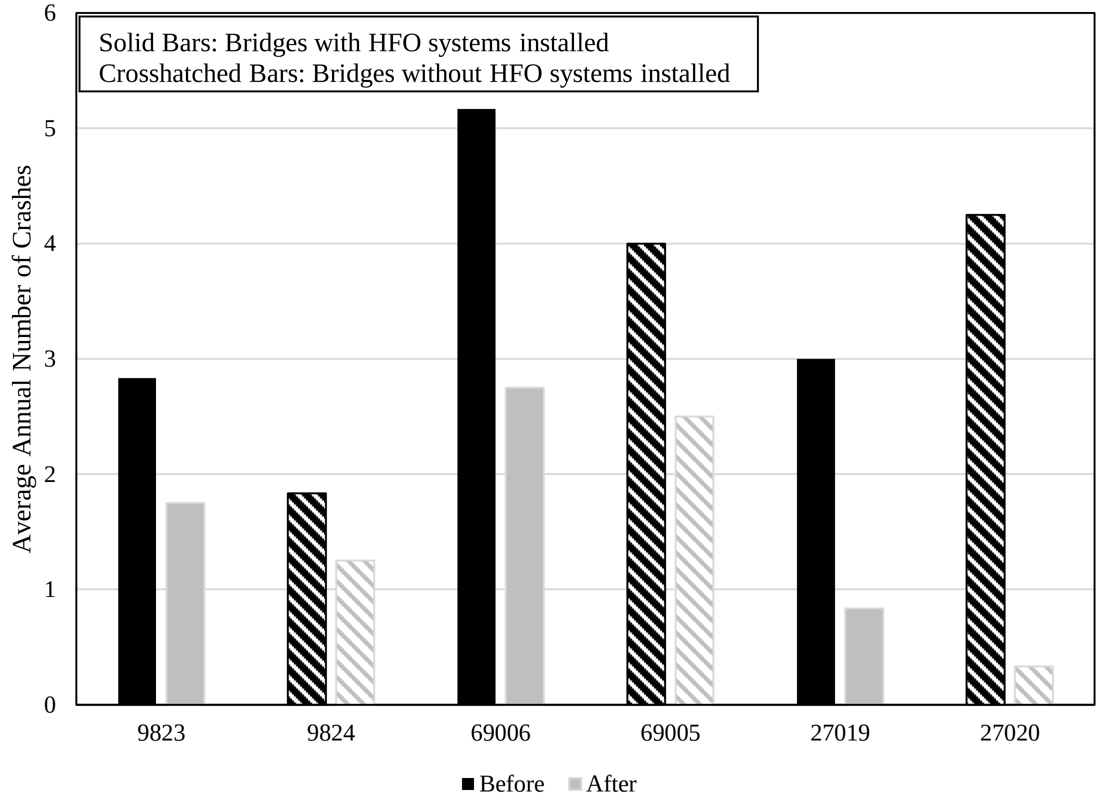


Figure 99: Side by side comparison of average number of crashes annually before and after the installation of HFO systems.

**Comparison of Crashes Normalized to Total Crashes in Minnesota**

Using data from the Minnesota Vehicle Crash Facts Reports for the years 2004 through 2012 the crashes for each bridge location were normalized to the total crashes occurring in the state. The percentages of crashes occurring on each bridge both before and after the installation of an HFO system are compared in Figure 100. For example on average 0.07% of crashes in Minnesota occurred annually on Bridge 9067 [Poly-Carb, Flint] while only 0.03% of crashes in Minnesota occurred on that bridge after the installation of an HFO system. Interestingly the bridges discussed earlier without HFO systems also saw a drop in the average percentage of Minnesota’s crashes. On average bridges with an HFO system saw a drop of 0.021% while bridges without an HFO system dropped 0.025%.

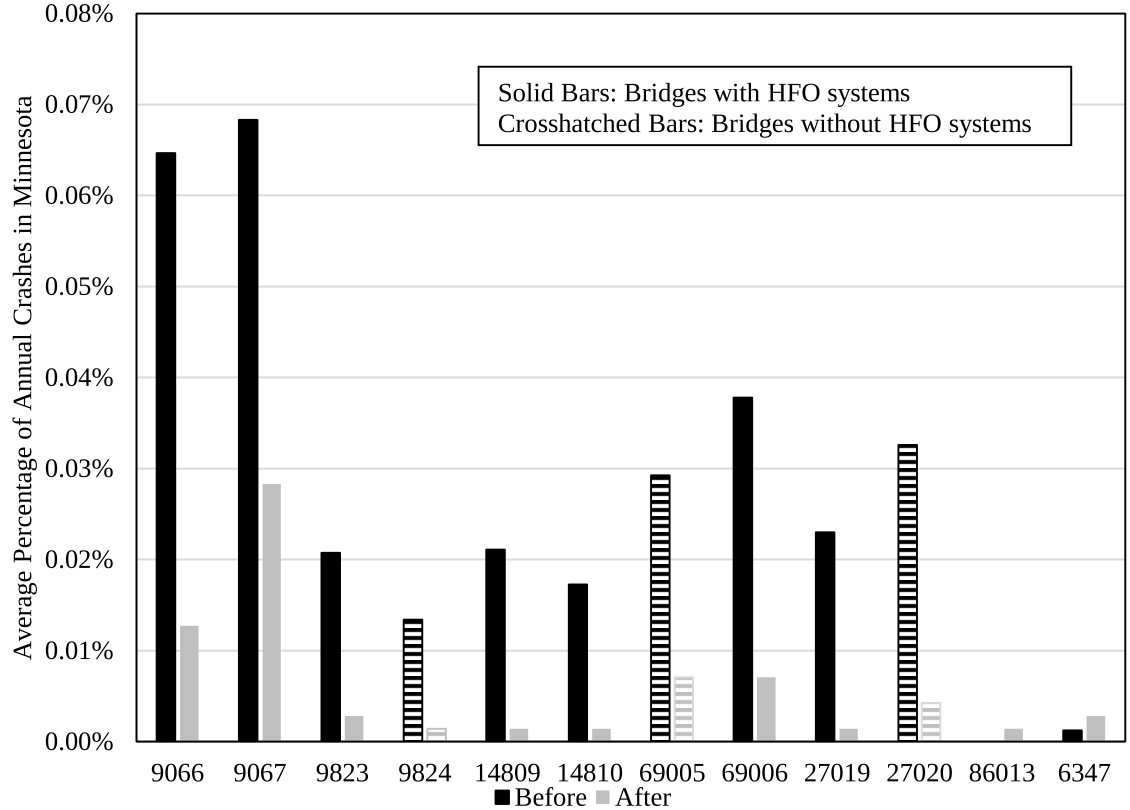


Figure 100: Average percentage of total Minnesotan crashes occurring on each bridge before and after installation of HFO systems.

During the years of this study the general trend across the state of Minnesota was a decrease in the total number of crashes. The crashes occurring on bridges in this study became a smaller percentage of the total crashes in the state, however rate the amount in which they dropped is comparable to bridge decks in the same location without an HFO. A comparison of Bridge 69005 and Bridge 69006 (Poly-Carb, Taconite) throughout the analyzed years is shown in Figure 101. The same decreasing trend in percentage of total statewide accidents is present for both bridges even after an epoxy system was installed on the deck of Bridge 69006 (Poly-Carb, Taconite).

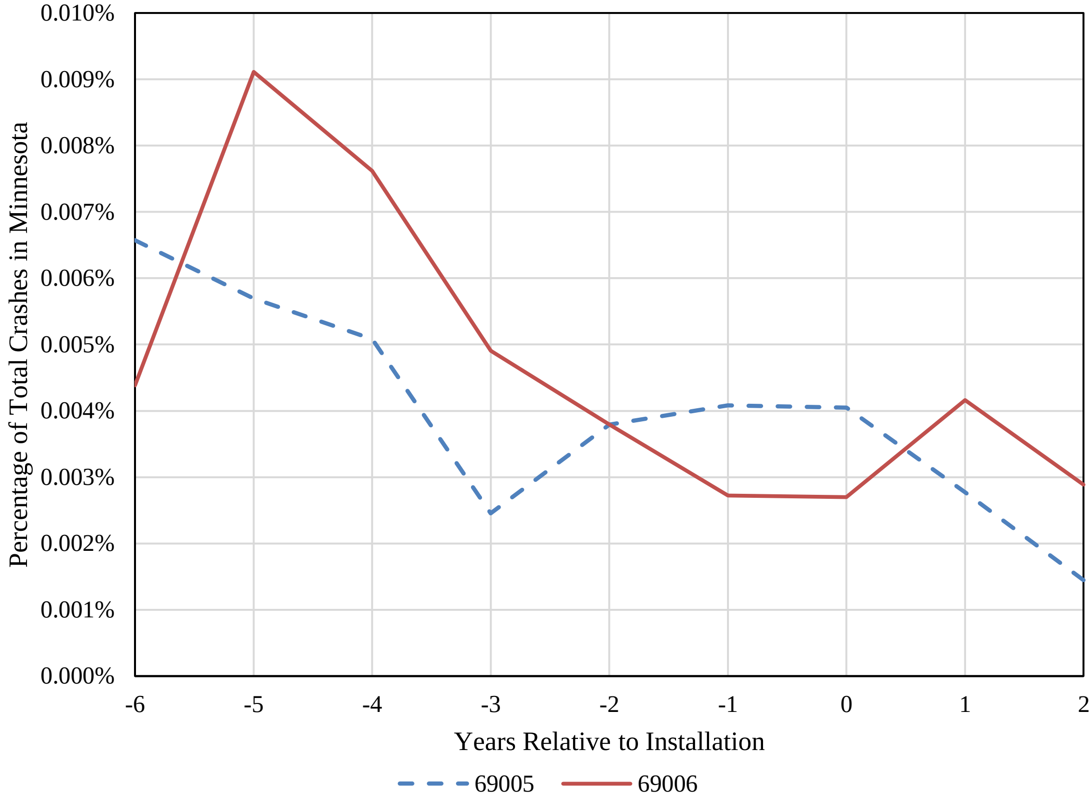


Figure 101: Side by side comparison of the percentage of total accidents in Minnesota across time.

Figure 100 and Figure 101 provide further evidence that although there is a decrease in crashes on bridges with HFO systems, the reduction does not conclusively appear to be the direct result of an HFO installation creating a perceived benefit of HFO systems.

**Comparison of Crashes with the Site's Surface Friction Values**

Since a minimum skid resistance value does not exist for crash reduction, it is possible to improve skid resistance of a bridge deck without actually improving the safety of the bridge. The following comparison will compare skid number (SN) values with the number of crashes per million vehicles at each bridge location (Figure 102). In this figure, the triangle markers indicate curved sections while tangent sections are identified with circle markers. The bridges are also split into three categories by speed limit: under 55 mph [89 km/h], 55 mph [89 km/h], and over 55 mph [89 km/h].

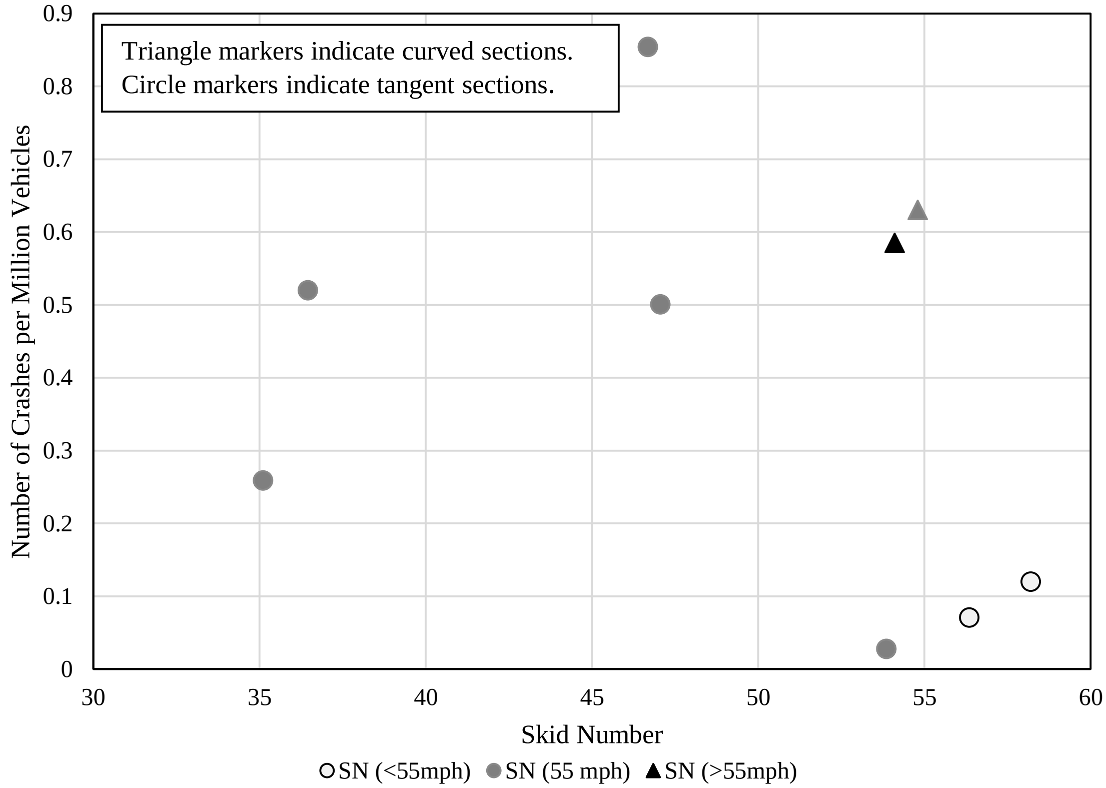


Figure 102: Skid Numbers versus the number of crashes per million vehicles.

Below in Figure 103, the same data as Figure 102 is displayed, except the x-axis has coefficient of friction values. Skid numbers are divided by 100 in order to put them in the coefficient of friction form. The coefficient of friction ($C_f$) values in the plot are results from the DFT testing in Chapter 5.

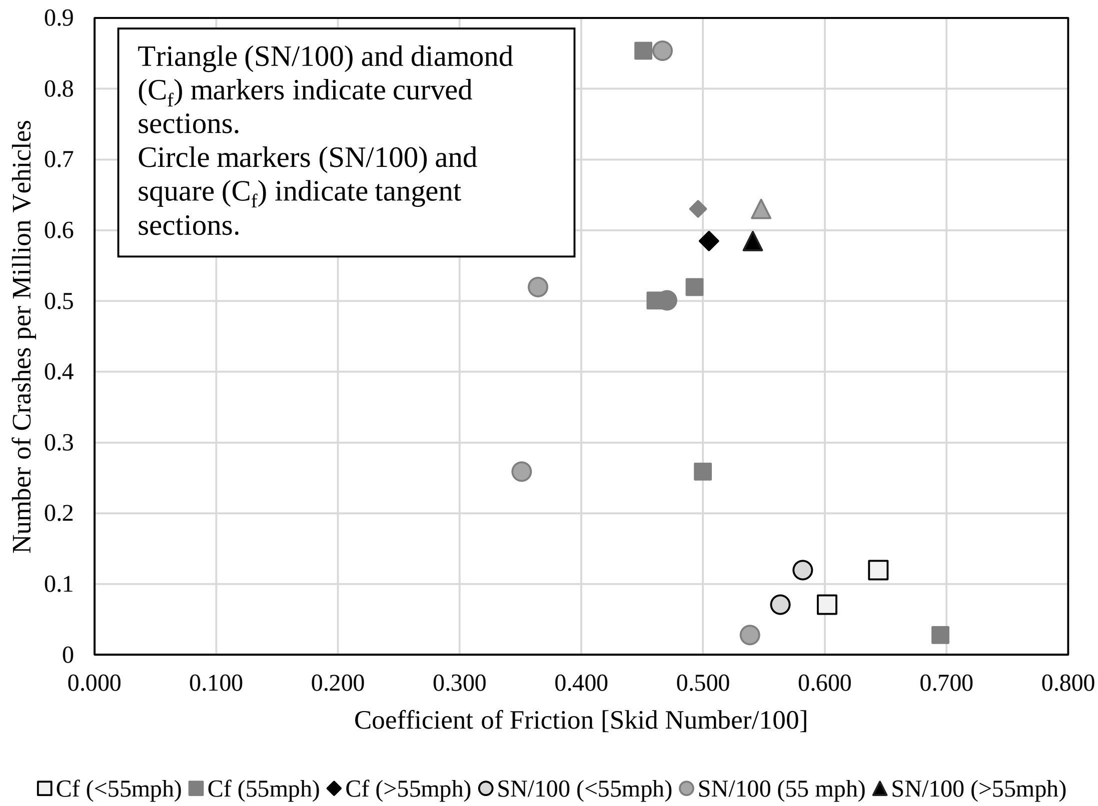


Figure 103: Coefficient of Friction and SN/100 versus the number of crashes per million vehicles

From this data, roads with lower speeds roads tend to have a lower crash rate, and curved sections have a larger crash compared to tangent sections which correlates with previous research (Long, 2014).

However when comparing the SN or $C_f$ values with crash rates there is not a noticeable trend. The number of crashes per million vehicles are not consistent when comparing different bridges with similar skid numbers. Furthermore there is not a consistently descending crash rate as the SN values increase suggesting that SN values are not a good indicator for crash rate reductions on bridges. This is confirmed by both types of testing lacking trends in their results.

## Benefit-Cost Ratio Analysis

A benefit-cost ratio analysis similar to previous research was conducted. The purpose of this analysis is to compare the benefits of HFO system's through a monetary lens. As discussed in the Literature Review, the severity of a crash can have an associated cost. The cost association created by MnDOT (Table 5) was used in this calculation. The average annual associated cost of crashes were found for each bridge before and after the installation of an HFO system. The average annual benefit is the difference between the average annual associated costs before and after installation. This average annual benefit was multiplied by four years and compared to the estimated cost of installation. Several reasons justify the use of four years for this analysis:

(1) The previous research predicted that the life expectancy of HFO systems in Minnesota was between three to five years.
(2) In Chapter 4 when looking at the skid numbers, several bridges fell below an SN of 50 around the fourth year in service.
(3) The average life of an HFO system in this study is 4.83 years.

Based on these factors, it is reasonable to place the effective lifespan of an HFO system between four to five years. Systems that exist past that time are not necessarily detrimental to the safety of a bridge deck's surface safety, but their benefits may be significantly compromised.

The benefits, costs, and benefit-cost (B/C) ratio are found below in Table 25, while a complete calculation is found in the Appendix. It should be noted that the estimated cost of installation is based on a unit price for the different HFO systems and an estimated surface area of the bridge. The cost is not based on any bid or construction data. A benefit-cost ratio larger than one means that the monetary benefit provided by an HFO system installation is greater than the cost of installation. Five of the nine bridges have a B/C ratio above one.

Table 25: Average Annual Benefits, Estimated Costs of HFO Installation, and Four Year Benefit-Cost Ratios

| Bridge | Average Annual Associated Cost of Crashes | | Average Annual Benefit | Estimated Cost of Installation | Average Four Year Benefit | Four Year B/C Ratio |
|---|---|---|---|---|---|---|
| | Before Installation | After Installation | | | | |
| 6347 | $586.67 | $11,085.71 | -$10,499.05 | $188,720.00 | -$41,996.19 | -0.22 |
| 9066 | $114,293.33 | $34,571.43 | $79,721.90 | $357,500.00 | $318,887.62 | 0.89 |
| 9067 | $141,520.00 | $138,514.29 | $3,005.71 | $357,500.00 | $12,022.86 | 0.03 |
| 9823 | $66,400.00 | $6,844.44 | $59,555.56 | $34,776.00 | $238,222.22 | 6.85 |
| 14809 | $828,800.00 | $559,261.54 | $269,538.46 | $35,119.98 | $1,078,153.85 | 30.70 |
| 14810 | $27,066.67 | $12,092.31 | $14,974.36 | $35,119.98 | $59,897.44 | 1.71 |
| 69006 | $135,692.31 | $34,488.89 | $101,203.42 | $35,857.92 | $404,813.68 | 11.29 |
| 27019 | $98,072.73 | $800.00 | $97,272.73 | $57,200.00 | $389,090.91 | 6.80 |
| 86013 | $0.00 | $11,890.91 | -$11,890.91 | $46,706.00 | -$47,563.64 | -1.02 |

A perceived benefit exists in these benefit-cost ratios similar to the perceived benefit discussed above when comparing side by side bridges and comparing the crashes normalized to total crashes in Minnesota. The reduction in total crashes over time reduces the cost associated to crashes. This benefit is also available in bridges without an HFO installation (Table 26). As shown below, two bridges without an HFO system have a reduction in costs associated with crashes. A benefit-cost ratio for these bridges cannot be calculated because the “cost” is zero.

Table 26: Average Annual Benefits of Bridges without an HFO System Installed

| Bridge | Average Annual Associated Cost of Crashes | | Average Annual Benefit |
|---|---|---|---|
| | Before Installation | After Installation | |
| 9824 | $33,569.23 | $4,888.89 | $28,680.34 |
| 69005 | $40,707.69 | $42,355.56 | -$1,647.86 |
| 27020 | $75,745.45 | $23,155.56 | $52,589.90 |

## Crash Analysis Summary

Effectiveness of crash prevention potential of high-friction overlay treatments in Minnesota was evaluated through analysis of crash and weather condition data. The data was provided by MnDOT from the Public Safety Database. The crash data compared in this paper occurred during the time frame from 2003 through 2013 and corresponded to eleven different bridges in this study. This analysis was completed to gain perspective on the performance of bridge decks with high-friction overlay treatments. The main concern for the wear courses assessed through this analysis was the additional safety provided due to the higher friction of such overlay systems. The use of high friction overlays is increasing in colder climate locations due to their perceived benefit of providing safer driving conditions during inclement weather.

On the basis of crash data analysis the following conclusions can be drawn:

- A majority of crashes occur in either dry or ice packed snow surface conditions. The friction properties of an HFO system have little effect in preventing crashes during such instances. Even bridges equipped with automated deicing systems had a significant number of ice packed snow surface condition crashes.
- The main purpose of an HFO is to provide higher SN values to facilitate safer roads and reduce crash rates. HFO systems do provide an increase in surface

friction, however noticeable trends between SN values and crash rates do not exist suggesting that skid numbers are not a reliable indicator of crash rate reductions.

- There was a negligible change in the distribution of crash severity after HFO systems were placed on the bridge decks.
- Five of the nine bridges had a benefit-cost (B/C) ratio over one, however the benefits cannot be directly attributed to the installation of an HFO system.
- Further research is recommended for efficient and effective removal of ice packed snow from bridge decks. The removal of such conditions would significantly help reduce the number of crashes on bridge decks in cold climates such as Minnesota.
- Although minimal crash prevention occurs, of the systems, the thin-bonded open graded asphalt overlay [Novachip] provided the best performance in crash reduction, however it was closely followed by two of the epoxy based systems [SafeLane and Poly-Carb].
- Of the Poly-Carb systems with different aggregates, the taconite aggregate showed best performance in reduction of crashes as compared to flint or basalt.
- Previous research has looked at crashes occurring before and after the installation of an HFO system. The results provided the perceived benefit of crash reduction, however this perceived benefit occurs on bridges without an HFO installation. The average number of crashes occurring on a bridge deck is reduced regardless of whether or not an HFO system was installed and at about the same rate for each location.

Overall crashes in the state of Minnesota are on a reducing trend, but the reduction in crashes on bridge decks with HFO systems cannot be completely attributed to the use of such systems.

The results and discussions presented herein are part of a larger encompassing study of HFO systems in Minnesota. Several other factors are being researched that attribute to the performance of the systems in the field. Based on this analysis high friction overlays are not the primary reason for a reduction in crashes when placed on a bridge, however HFO systems do have other tangible benefits. HFO systems provide a strong waterproof seal over bridge decks reducing chloride ion penetration and thus mitigating corrosion of reinforcing steel. Other aspects such as geometrics play an important role in crashes and should be incorporated into future studies.

# Chapter 6: Summary, Conclusions, and Recommendations

## Summary

This study was proposed to continue the investigation of high friction overlay (HFO) systems in Minnesota, in which the Minnesota Department of Transportation (MnDOT) has invested. The investments were in efforts to provide safer surface conditions on bridge decks. The recommendation for further investigation came after a study and report of SafeLane HFO products placed in Minnesota published by MnDOT (Evans, 2010). A larger scope of study was possible due to the continued use of HFO systems in Minnesota. Four different types of proprietary HFO systems were investigated on fourteen different bridges. This study encompassed more bridges, more testing, and longer time intervals since the installation of the HFO systems than both the predeceasing research, and research previously conducted in other states.

A comparative analysis of the different HFO systems was completed with the purpose of investigating the HFO systems in regard to their field performance and the safety benefits they provide. A second objective of this study is to serve as a model for future research involving HFO systems and their benefits in the reduction of crashes and ultimately the safety of roadways they are placed on. Through the observation, testing, and data analysis of different conditions and performances for each HFO system the study looked to answer several question regarding HFO systems including:

(1) How do HFO systems perform in cold weather climates that exist in Minnesota?
(2) Do HFO systems provide benefits that reduce crash rates on bridges?
(3) Is there is a correlation between the SN values of an HFO system and their crash rates.
(4) Do the benefits HFO systems provide to Minnesota bridge decks justify their use in the state?

## Conclusions

High friction overlay (HFO) systems are marketed to provide three main benefits for infrastructure they are placed on: (1) high friction, (2) an impermeable surface, and (3) crash rate reduction.

HFO Systems provide high friction values at the time of installation however those values steadily decrease over time. Different bridge sites are performing at varying states of satisfaction. Aggregate raveling, heavy abrasion, and aggregate polishing are three distresses that exist on every bridge deck. These distresses are mostly caused due to the heavy use of snow plows for winter maintenance in the cold climate and ultimately reduce the systems' surface friction.

- Novachip systems perform better in regards to abrasion. The ductility of asphalt allows for the continual embedment of aggregates into the matrix which helps to reduce the amount of aggregate loss.
- Poly-Carb systems' mean texture depth values (measure of surface macro-texture) are reduced on average by 6-8% a year.
- SafeLane systems' MTD values are reduced on average by 5% a year.
- Significant abrasion, especially in the non-wheel path, occurred on the Poly-Carb and SafeLane bridges when comparing their MTD values from 2013 to 2014.

The significant high friction values of HFO systems are limited in cold climates due to their harsh maintenance operations which rapidly deteriorate the high friction surfaces. The lifespan of an HFO system in Minnesota is only about four or five years. After this time friction values of HFO systems are very similar to those provided by conventional concrete or asphalt overlays.

HFO systems provide an impermeable surface, but only provided the systems do not become distressed enough where delamination, cracking, or abrasion to the bridge deck occurs.

Of the fourteen bridges in this study:

- Two bridges had delaminated HFO systems (Br 4190 and 9036, both Poly-Carb)
- Three bridges had cracks in the HFO systems (Br 27019, 86013, and 86019 - all Novachip)
- One bridge had delamination an cracking (Br 6347, Transpo)
- One bridge had cracking and complete removal system sections due to abrasion (Br 27758, Transpo)

Thus fifty percent of the bridges in this study have compromised systems where water and chloride ions are able to penetrate into the bridge deck. Through the chloride ion penetration testing, it was found that when properly placed and when compromising distresses have not occurred, HFO systems have the ability to protect infrastructure from chloride ion penetration. However, distresses of the system such as delamination, abrasion, or cracking, mitigate this protection in a localized area near the distress reducing the infrastructure protection effectiveness of an HFO system.

The main concern for the wear courses assessed through this analysis was the additional safety provided due to the higher friction of such overlay systems. The use of high friction overlays is increasing in colder climate locations, however they have a perceived benefit of providing safer driving conditions during inclement weather.

- Ice packed snow surface conditions create situations where the friction properties of an HFO system have little effect in preventing crashes. Even bridges equipped with automated deicing systems had a significant number of ice packed snow surface condition crashes.
- The main purpose of an HFO is to provide higher SN values to facilitate safer roads and reduce crash rates. HFO systems do provide an increase in surface friction, however noticeable trends between SN values and crash rates do not exist suggesting that skid numbers are not a reliable indicator of crash rate reductions.
- There was a negligible change in the distribution of crash severity after HFO systems were placed on the bridge decks.

- Five of the nine bridges had a benefit-cost (B/C) ratio over one, however the benefits cannot be directly attributed to the installation of an HFO system.
- Previous research has looked at crashes occurring before and after the installation of an HFO system. The results provided the perceived benefit of crash reduction, however this perceived benefit occurs on bridges without an HFO installation. The average number of crashes occurring on a bridge deck is reduced regardless of whether or not an HFO system was installed and at about the same rate for each location.

Overall crashes in the state of Minnesota are on a reducing trend, but the reduction in crashes on bridge decks with HFO systems cannot be completely attributed to the use of such systems.

Of the different aggregates tested, Flint aggregate is the least desirable aggregate. Although it had comparable performances with the other aggregates in the laboratory testing, Flint aggregates had the worst performance in the field. Poly-Carb bridges with a flint aggregate (Br 9066 and 9067) had some of the lowest MTD values and coefficients of friction. The bridges also had a significant amount of polished areas and aggregate removal. The Transpo system with Flint (Br 6347) had a significant amount of polished areas where too much epoxy was applied which may have been caused by the flat and elongated characteristics of the aggregate. The particles required less epoxy to be covered and when this adjustment was not made polished areas were created.

Of the four different systems in this study, Novachip systems appear to have the best performance in the field. Novachip systems are constructed similar to asphalt overlays, thus the system is compacted with a roller during construction. This compaction helps to push the aggregates further into the asphalt matrix and achieve a higher bond. Additionally, asphalt remains pliable while it ages. Aggregate in the Novachip systems are able to compress into the asphalt under a load even after the initial placement.

The elasticity of asphalt allows the system to absorb some of the energy caused by abrasion. Epoxy systems cannot compress and cannot absorb the energy as the stiffness of epoxy is very high. This results in the applied forces breaking the bonds formed between the epoxy and aggregates and removing aggregates from the system. Thus Novachip systems are superior compared to epoxy systems in aggregate retention. This aggregate retention also helps to maintain a high friction throughout the system's service life. The two main distresses occurring in the Novachip systems were reflective cracking and snowplow abrasion. Both the reflective cracks and areas of extensive snow plow abrasion came from settlement in the approach panels or sections of the bridge. Thus the major distresses are preventable by placing Novachip systems on well-functioning bridges. The negative aspect of Novachip systems is that asphalt is not a completely impervious surface like hardened epoxy, so Novachip systems have a lower potential for infrastructure protection.

## Recommendations

Common surface conditions during the winter such as ice packed snow surfaces, the presence of water, and winter maintenance practices with snow plows are all detrimental to an HFO system's performance in cold weather climates. A life span of only four or five years exists due to distresses that occur on HFO systems caused by common winter conditions. These distresses severely mitigate the benefits HFO systems provide. Furthermore, at an average cost of around $5.50 per square foot, the higher costing HFO systems also do not provide an economical benefit. Since the friction and infrastructure protection benefits are not long-lasting, or are compromised due to distresses, a crash reduction does not occur as a direct result of HFO systems, and the systems come at a higher cost than conventional overlays, it is recommended that HFO systems are not used in Minnesota.

HFO systems in mild climates may provide more benefits because: (1) their friction values may not be compromised due to a lesser use of snow plows, and (2) ice packed snow conditions, which were present for 39% of the crashes in this study, are not as prevalent in mild climate areas. Other cold climate region transportation agencies should conduct similar studies to justify the use of HFO systems. Most notably, the crash rate reductions on HFO systems should be compared to the overall crash rate reduction in their jurisdiction. Looking solely at the crashes before and after the installation of an HFO system provides the perceived benefit that a reduction in crashes is occurring if the trend of overall crashes is also decreasing.

Although this research shows that a reduction in crashes is not occurring due to the installation of HFO systems, it cannot conclusively state whether HFO systems in fact increase the crash rate potential on a bridge. This potential increase in crash rates is suggested by data in this research such as: (1) an increase in crashes during dry surface conditions, (2) a significant number of crashes occurring during ice packed snow conditions along with the larger mean texture depth which may facilitate ice packed snow conditions to occur, and (3) an increase in polished or raveled areas as systems age. It is recommended that further research looks more in depth into the possibility of HFO systems having adverse effects on crash rates.

Overall the DFT and skid trailer tests provide similar surface friction values. It is recommended that DFT testing becomes a more prevalent test method because of the benefits it provides compared to the skid trailer. It allows for multiple tests run in the same location providing higher precision. It also eliminates error that may interfere with test results such as bouncing, lateral movement, improper water spray film thickness, or incorrect speeds of the skid trailer which must be in motion for its testing procedure. The fact that testing is done in a static location for DFT testing provides the opportunity to test more accurately in wheel and non-wheel paths. It also provides the opportunity to test specific distressed areas to compare with non-distressed areas.

Several recommendations can be made for research into the improvement of HFO products. Research on construction practices, and education of contractors should be conducted in order to provide a more consistent finished product. Improper levels of epoxy during the placement of HFO systems can lead to polished areas from excessive epoxy, or aggregate raveling due to an insignificant amount of placed epoxy. HFO systems must be placed more consistently in order to improve the consistency of their friction values. Research should be conducted into a mechanized placement system for epoxy systems, the research should look mostly into how a machine could improve the consistency of construction, and if such a machine has a cost benefit.

Based on comparisons between asphalt based and epoxy based HFO systems. Companies that produce epoxy based systems should research more ductile epoxies. Increasing the ductility may allow for absorption of abrasion loads. This absorption may help to reduce the amount of aggregate raveling occurring in epoxy based systems. This ductility however must not compromise the overall bond strength between the HFO system and the underlying concrete deck.

It appears that as the mean texture depth (MTD) of an HFO system increases, so does the coefficient of friction. However faster rates at which the coefficient of friction increases occur with systems that use a nominal maximum aggregate size (NMAS) of 0.187 inches [5 mm] compared to 0.375 inches [9.5 mm]. Further research should be conducted to verify if the use of 0.187 inch [5 mm] aggregates is more beneficial.

Testing of the complete HFO systems in a laboratory setting should be researched. Material testing such as impact, abrasion, and bending strength testing of HFO systems in a laboratory setting may provide insight on ways to improve HFO systems and could also aid in the study of previously recommended improvements.

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# Appendix

## Chapter 3 Appendix

This section provides raw data and calculations used to complete the results section in Chapter 3.

**Density, Specific Gravity, & Absorption (Fine Aggregate) Data**

The results in Table 9 and Table 10 (See Results section) were calculated using Equations 1-6 and the Data found in Table 27. Data from Table 28 and equation 7 were used to calculate the values found in Table 11 of the results section.

Table 27: Specimen Mass Data

| | Basalt | Flint | Taconite A | Taconite B | SafeLane |
|---|---|---|---|---|---|
| A | 481 | 463.6 | 474.4 | 476.8 | 475.8 |
| B | 666.2 | 666.2 | 666.2 | 666.2 | 666.2 |
| C | 972.5 | 960.1 | 990.8 | 987.1 | 976.8 |
| S | 501.7 | 501.9 | 500.2 | 500.9 | 501.8 |

A= Mass of oven dry specimen (g)

B= Mass of pycnometer filled with water (g)

C= Mass of pycnometer filled with specimen and water (g)

S= Mass of saturated surface-dry specimen (g)

$$\text{Oven Dry Density (kg/m3)} = 997.5 * \left[\frac{A}{(B+S-C)}\right] \quad (1)$$

$$\text{SSD Density (kg/m3)} = 997.5 * \left[\frac{S}{(B+S-C)}\right] \quad (2)$$

$$\text{Apparent Density (kg/m3)} = 997.5 * \left[\frac{A}{(B+A-C)}\right] \quad (3)$$

$$\text{Oven Dry Relative Density (Specific Gravity)} = 997.5 * \left[\frac{A}{(B+S-C)}\right] \quad (4)$$

$$\text{SSD Relative Density (Specific Gravity)} = 997.5 * \left[\frac{S}{(B+S-C)}\right] \quad (5)$$

$$\text{Apparent Relative Density (Specific Gravity)} = 997.5 * \left[\frac{A}{(B+A-C)}\right] \quad (6)$$

Table 28: Absorption Data

| Aggregate | Initial Mass (g) | SSD Mass (g) |
|---|---|---|
| Basalt | 2212.5 | 2243 |
| Flint | 2138 | 2167 |
| Taconite A | 2346.5 | 2374.5 |
| Taconite B | 2624.5 | 2652.5 |
| SafeLane | 2450 | 2486 |

$$\text{Absorption (\%)} = 100 * \left[\frac{(Mass_{SSD} - Mass_{dry})}{Mass_{SSD}}\right] \quad (7)$$

**Resistance to Degradation: Los Angeles Machine Data**

The results in Table 12 (See Results section) were calculated using the data from Table 29 in Equations 8.

Table 29: Los Angeles Machine Degradation (Test Samples of Grading D)

| Aggregate | Mass Before (g) | Mass After (g) |
|---|---|---|
| Basalt | 4972.1 | 4816.83 |
| Flint | 4982.47 | 4831.12 |
| SafeLane | 4966.58 | 4736.84 |
| Taconite  A | 4975.51 | 4818.06 |
| Taconite B | 4979.62 | 4778.32 |

$$\text{Percent Loss by Abrasion} = 100 * \left[\frac{\text{Mass After} - \text{Mass Before}}{\text{Mass Before}}\right] \quad (8)$$

**Uncompacted Voids Data**

The results in Table 13 (See Results section) were calculated using the data from Table 30 in Equations 9-10.

Table 30: Uncompacted Voids Data

| | | | | | |
|---|---|---|---|---|---|
| V | 100 | 100 | 100 | 100 | 100 |
| E | 189.6 | 189.6 | 189.6 | 189.6 | 189.6 |
| T1 | 355.2 | 330.8 | 353.1 | 334.2 | 340.6 |
| T2 | 355.7 | 332 | 353.2 | 333 | 340.8 |
| T3 | 355.4 | 331.4 | 353.3 | 332.3 | 340.7 |
| Ta | 355.4 | 331.4 | 353.2 | 333.2 | 340.7 |
| F | 165.8 | 141.8 | 163.6 | 143.6 | 151.1 |
| $G_s$ | 2.65 | 2.23 | 2.7 | 2.49 | 2.46 |
| **U** | **37.40%** | **36.40%** | **39.40%** | **42.30%** | **38.60%** |

Where V= Volume of cylindrical measure (ml)

E= Mass of cylindrical measure (g)

T= Gross mass of filled cylindrical measure (g)

F= Net mass of fine aggregate (g)

$G_s$= Dry relative density (specific gravity)

U= Uncompacted voids in the material (%)

$$F = T_a - E \tag{9}$$

$$U = 100*\left[\frac{V-\left(\frac{F}{G_s}\right)}{V}\right] \tag{10}$$

**Flat/Elongated Particles**

The data shown below in Table 31 thru Table 34 are summarized for comparison in Figure 17 through Figure 19 (See Results Section).

Table 31: Basalt - Flat/Elongated Particles

| Flat/Elongated Ratio | Number of Particles and Percentage | | | | | |
|---|---|---|---|---|---|---|
| | 2 to 1 | | 3 to 1 | | 5 to 1 | |
| Flat | 30 | 28% | 4 | 4% | 0 | 0% |
| Elongated | 2 | 2% | 0 | 0% | 0 | 0% |
| Flat/Elongated | 1 | 1% | 0 | 0% | 0 | 0% |
| None | 75 | 69% | 104 | 96% | 108 | 100% |
| **Total** | **108** | **100%** | **108** | **100%** | **108** | **100%** |

Table 32: Taconite A- Flat/Elongated Particles

| Flat/Elongated Ratio | Number of Particles and Percentage | | | | | |
|---|---|---|---|---|---|---|
| | 2 to 1 | | 3 to 1 | | 5 to 1 | |
| Flat | 68 | 58% | 28 | 24% | 4 | 3% |
| Elongated | 4 | 3% | 0 | 0% | 0 | 0% |
| Flat/Elongated | 3 | 3% | 0 | 0% | 0 | 0% |
| None | 42 | 36% | 89 | 76% | 113 | 97% |
| **Total** | **117** | **100%** | **117** | **100%** | **117** | **100%** |

Table 33: Taconite B- Flat/Elongated Particles

| Flat/Elongated Ratio | Number of Particles and Percentage | | | | | |
|---|---|---|---|---|---|---|
| | 2 to 1 | | 3 to 1 | | 5 to 1 | |
| Flat | 72 | 57% | 21 | 17% | 1 | 1% |
| Elongated | 5 | 4% | 0 | 0% | 0 | 0% |
| Flat/Elongated | 2 | 2% | 0 | 0% | 0 | 0% |
| None | 47 | 37% | 105 | 83% | 125 | 99% |
| **Total** | **126** | **100%** | **126** | **100%** | **126** | **100%** |

Table 34: SafeLane- Flat/Elongated Particles

| Flat/Elongated Ratio | Number of Particles and Percentage | | | | | |
|---|---|---|---|---|---|---|
| | 2 to 1 | | 3 to 1 | | 5 to 1 | |
| Flat | 46 | 43% | 11 | 10% | 0 | 0% |
| Elongated | 5 | 5% | 0 | 0% | 0 | 0% |
| Flat/Elongated | 2 | 2% | 0 | 0% | 0 | 0% |
| None | 54 | 50% | 96 | 90% | 107 | 100% |
| **Total** | **107** | **100%** | **107** | **100%** | **107** | **100%** |

**Soundness of Aggregate: Freeze Thaw Data**

The results in Figure 8 (See Results section) were calculated using the data from Table 35 in Equations 11.

Table 35: Freeze Thaw Resistance Data

| Sample (Sieve Size) | Mass of Water (g) | Mass of Salt (g) | Aggregate Mass (g) | | Loss of Mass (g) | Percent of Mass Lost (%) |
|---|---|---|---|---|---|---|
| | | | Original | Final | | |
| Basalt #8 | 946 | 28.4 | 150 | 146.5 | 3.5 | 2.3 |
| Flint #8 | 959 | 28.8 | 150 | 147 | 3 | 2.0 |
| Flint #16 | 949 | 28.5 | 150 | 146.5 | 3.5 | 2.3 |
| Taconite A #8 | 945 | 28.4 | 150 | 145.5 | 4.5 | 3.0 |
| Taconite A #16 | 982 | 29.5 | 150 | 147 | 3 | 2.0 |
| Taconite B #8 | 988 | 29.6 | 150 | 148.5 | 1.5 | 1.0 |
| Taconite B #16 | 947 | 28.4 | 150 | 148.5 | 1.5 | 1.0 |
| Safe Lane # 4 | 980 | 29.4 | 150 | 147.5 | 2.5 | 1.7 |
| Safe Lane #8 | 951 | 28.5 | 150 | 149.5 | 0.5 | 0.3 |

Note: Flint #8 stands for Flint retained on the #8 sieve

$$\text{Percent of Mass Lost (\%)} = 100 * \left[\frac{\text{Loss of Mass (g)}}{\text{Aggregate Original Mass (g)}}\right] \tag{11}$$

**Sieve Analysis**

Below are the gradations (Table 36 through Table 40) for the five aggregate samples. The gradations are summarized in the Chapter 3 Results section in Figure 22.

Table 36: Basalt Sieve Analysis

| Sieve Size | Mass Retained (g) | Cumulative Mass (g) | Percent Passing (%) |
|---|---|---|---|
| 0.375 inch | 0 | 0 | 100.00% |
| #4 | 21.4 | 21.4 | 98.53% |
| #8 | 1368.4 | 1389.8 | 4.44% |
| #16 | 64.2 | 1454 | 0.02% |
| #50 | 0.2 | 1454.2 | 0.01% |
| #100 | 0 | 1454.2 | 0.01% |
| #200 | 0.1 | 1454.3 | 0.00% |
| Pan | 0 | 1454.3 | 0.00% |
| Total | 1454.3 | | |

Table 37: Flint Sieve Analysis

| Sieve Size | Mass Retained (g) | Cumulative Mass (g) | Percent Passing (%) |
|---|---|---|---|
| 0.375 inch | 0 | 0 | 100.00% |
| #4 | 1.4 | 1.4 | 99.92% |
| #8 | 1266.4 | 1267.8 | 29.40% |
| #16 | 519.6 | 1787.4 | 0.47% |
| #50 | 7.6 | 1795 | 0.04% |
| #100 | 0.2 | 1795.2 | 0.03% |
| #200 | 0.2 | 1795.4 | 0.02% |
| Pan | 0.4 | 1795.8 | 0.00% |
| Total | 1795.8 | | |

Table 38: Taconite A Sieve Analysis

| Sieve Size | Mass Retained (g) | Cumulative Mass (g) | Percent Passing (%) |
|---|---|---|---|
| 0.375 inch | 0 | 0 | 100.00% |
| #4 | 69.4 | 69.4 | 95.82% |
| #8 | 1320 | 1389.4 | 16.30% |
| #16 | 269.4 | 1658.8 | 0.07% |
| #50 | 0.3 | 1659.1 | 0.05% |
| #100 | 0.1 | 1659.2 | 0.04% |
| #200 | 0.1 | 1659.3 | 0.04% |
| Pan | 0.6 | 1659.9 | 0.00% |
| Total | 1659.9 | | |

Table 39: Taconite B Sieve Analysis

| Sieve Size | Mass Retained (g) | Cumulative Mass (g) | Percent Passing (%) |
|---|---|---|---|
| 9.5 | 0 | 0 | 100.00% |
| #4 | 28 | 28 | 98.95% |
| #8 | 2018.5 | 2046.5 | 23.50% |
| #16 | 621.6 | 2668.1 | 0.26% |
| #30 | 4.1 | 2672.2 | 0.11% |
| #100 | 1.8 | 2674 | 0.04% |
| #200 | 0.7 | 2674.7 | 0.01% |
| Pan | 0.4 | 2675.1 | 0.00% |
| Total | 2675.1 | | |

Table 40: SafeLane Sieve Analysis

| Sieve Size | Mass Retained (g) | Cumulative Mass (g) | Percent Passing (%) |
|---|---|---|---|
| 9.5 | 0 | 0 | 100.00% |
| #4 | 847 | 847 | 61.36% |
| #8 | 1282.8 | 2129.8 | 2.84% |
| #100 | 56 | 2185.8 | 0.28% |
| #200 | 1.2 | 2187 | 0.23% |
| Pan | 5 | 2192 | 0.00% |
| Total | 2192 | | |

# Chapter 4 Appendix

This section provides additional equations, data, and plots used to complete Chapter 4.

### Mean Texture Depth

Equation 12 is used to find the Mean Texture Depth (MTD) as described in ASTM E965 – 96.

$$\text{MTD} = \frac{4V}{\pi D^2} \tag{12}$$

Where MTD = Mean Texture Depth (in)

V = sample volume ($in^3$)

D = average diameter of the area covered by material (in)

#### ***Sample Calculation using Test from Bridge 9823 (Atkinson, MN)***

100 cubic centimeters of sand was used in the test with the resulting 4 diameter measurements of 13.25”, 13.5”, 13.5”, and 13.75” for an average diameter of 13.5”.

$$\text{MTD} = \frac{4V}{\pi D^2} = \frac{4*\left(100\ cm^3 * \left[\frac{1\ in}{2.54\ cm}\right]^3\right)}{\pi*(13.5\ in)^2} = 0.0426\ \text{in}$$

### Coefficient of Permeability

Equation 13 is used to find the coefficient of permeability as described by the NCAT Asphalt Field Permeameter Kit AP-1B operating manual.

$$K = \frac{\left(\frac{a*L}{A*t}\right)}{\ln\left(\frac{h_1}{h_2}\right)} \tag{13}$$

Where K = Coefficient of permeability

a = 2.85 $cm^2$ = Inside cross sectional area of standpipe

L = 0.9525 cm = Length of sample (Thickness of epoxy system)

A = 167.53 $cm^2$ = Cross-sectional area of Permeameter

t = Elapsed time between $h_1$ and $h_2$

$h_1$ = Initial head, cm

$h_2$ = Final head, cm

***Sample calculation using data from a test on Bridge 9067***

$$K = \frac{\left(\frac{a*L}{A*t}\right)}{\ln\left(\frac{h_1}{h_2}\right)} = \frac{\left(\frac{2.85\ cm^2*0.9525\ cm}{167.53\ cm^2*600\ sec}\right)}{\ln\left(\frac{59\ cm}{58.5\ cm}\right)} = 2.26\text{x}10^{-7}\ \text{cm/sec}$$

## Friction Testing

The following data provides the average skid number results throughout the years of testing conducted by MnDOT (Table 41).

Table 41: Average Skid Number Test Results

| Date | | 2007 | 2008 | 2009 | 2010 | 2011 | 2012 | 2013 | 2014 |
|---|---|---|---|---|---|---|---|---|---|
| Bridge | 6347 | 0 | 0 | 0 | 0 | 0 | 0 | 58.24 | 54.8 |
| | 9066 | 0 | 0 | 0 | 120 | 52.25 | 47.05 | 41.68 | 0 |
| | 9067 | 0 | 0 | 0 | 120 | 0 | 46.68 | 44 | 0 |
| | 9823 | 0 | 0 | N/A* | 64.9 | 60.25 | 54.1 | 0 | 0 |
| | 14809 | 63.5 | 50.3 | 47.95 | 45.6 | 38.1 | 36.45 | 0 | 0 |
| | 14810 | 65.2 | 50.2 | 46.27 | 40.03 | 38.73 | 35.1 | 0 | 0 |
| | 27019 | 0 | 0 | 56.6 | 57.1 | 57.1 | 53.85 | 54.5 | 0 |
| | 27758 | 0 | 0 | 0 | 59.8 | 48.7 | 50.2 | 54.67 | 54.1 |
| | 69006 | 0 | 0 | 120 | 61.3 | 61.7 | 54.73 | 0 | 0 |
| | 86013 | 0 | 0 | 0 | 0 | 0 | 0 | 56.35 | 0 |
| | 86019 | 0 | 0 | 0 | 0 | 0 | 0 | 56.05 | 0 |
| | 9036 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 57.9 |

Note:

0 = No friction test was conducted

N/A* = a test was conducted but resulted in poor test data or a faulty test

**Chloride Ion Penetration**

Data for Bridge 9823 is found in Table 42 and Table 43, while data for Bridge 69006 is found in Table 44 through Table 46.

Table 42: Chloride Content on Bridge 9823 (Tested Oct 11, 2011)

| Depth (in) | Chloride Content (ppm Cl) | | | | | | |
|---|---|---|---|---|---|---|---|
| | Core 1 | Core 2 | Core 3 | Core 4 | Core 5 | Core 6 | Average |
| | Driving Lane | Driving Lane | Driving Lane | Passing Lane | Passing Lane | Passing Lane | |
| 0-1 | 5,382 | 6,849 | 5,118 | 4,869 | 4,139 | 5,676 | 5,339 |
| 1-2 | 3,828 | 4,309 | 2,082 | 1,483 | 2,853 | 4,038 | 3,099 |
| 2-3 | 3,063 | 2,895 | 140 | 116 | 2,459 | 3,792 | 2,078 |
| 3-4 | 3,074 | 2,007 | 116 | N/A | N/A | N/A | 1,732 |

Table 43: Chloride Content on Bridge 9823 (Tested Dec 1, 2014)

| Depth (in) | Chloride Content (ppm Cl) | | |
|---|---|---|---|
| | Core 1 | Core 2 | Average |
| | Driving Lane | Passing Lane | |
| 0-0.5 | 4,106 | 4,655 | 4,381 |
| 0.5-1 | 3,309 | 5,437 | 4,373 |
| 1-1.5 | 2,377 | 3,552 | 2,965 |
| 1.5-2 | 1,642 | 2,776 | 2,209 |
| 2-3 | 758 | 1,436 | 1,097 |
| 3-4 | 466 | 321 | 394 |
| 4-5 | N/A | 300 | 300 |

Table 44: Chloride Content on Bridge 69006 (Tested Oct 15, 2010)

| Depth (in) | Chloride Content (ppm Cl) | | | | | | |
|---|---|---|---|---|---|---|---|
| | Core 1 | Core 2 | Core 3 | Core 4 | Core 5 | Core 6 | Average |
| | Driving Lane | Driving Lane | Driving Lane | Passing Lane | Passing Lane | Passing Lane | |
| 0-1 | 4,028 | 4,453 | 3,805 | 3,972 | 4,614 | 4,175 | 4,175 |
| 1-2 | 2,330 | 2,486 | 2,476 | 1,078 | 2,078 | 1,968 | 2,069 |
| 2-3 | 1,099 | 1,404 | 1,341 | 907 | 366 | 622 | 957 |
| 3-4 | 812 | 582 | 839 | 1,351 | 1,563 | 785 | 989 |
| 4-5 | 482 | 226 | 438 | 1,122 | 432 | 721 | 570 |
| 5-6 | 384 | 241 | 116 | 953 | 117 | 298 | 352 |

Table 45: Chloride Content on Bridge 69006 (Tested Sep 5, 2011)

| Depth (in) | Chloride Content (ppm Cl) | | | | | | |
|---|---|---|---|---|---|---|---|
| | Core 1 | Core 2 | Core 3 | Core 4 | Core 5 | Core 6 | Average |
| | Driving Lane | Driving Lane | Driving Lane | Passing Lane | Passing Lane | Passing Lane | |
| 0-1 | 4,340 | 4,172 | 5,025 | 4,072 | 4,738 | 3,745 | 4,349 |
| 1-2 | 1,676 | 2,788 | 2,263 | 1,560 | 2,353 | 2,445 | 2,181 |
| 2-3 | 830 | 564 | 487 | 948 | 1,208 | 1,731 | 961 |
| 3-4 | N/A | N/A | N/A | N/A | N/A | 1,981 | 1,981 |

Table 46: Chloride Content on Bridge 69006 (Tested Dec 1, 2014)

| Depth (in) | Chloride Content (ppm Cl) | | |
|---|---|---|---|
| | Core 1 | Core 2 | Average |
| | Driving Lane | Passing Lane | |
| 0-0.5 | 4,338 | 4,472 | 4,405 |
| 0.5-1 | 4,116 | 5,108 | 4,612 |
| 1-1.5 | 3,176 | 4,406 | 3,791 |
| 1.5-2 | 1,922 | 3,845 | 2,884 |
| 2-3 | 654 | 3,357 | 2,006 |
| 3-4 | 1,062 | 3,142 | 2,102 |
| 4-5 | N/A | 2,530 | 2,530 |

# Chapter 5 Appendix

## Data for Contributing Factors in Crashes

The following provides the number of crashes with each contributing factor by bridge.

Table 47: Data for Contributing Factors

| Contributing Factor | Number of Crashes by Bridge per Contributing Factor | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | 6347 | 9066 | 9067 | 9823 | 14809 | 14810 | 27019 | 69006 | 86013 |
| Not Specified | 0 | 14 | 19 | 2 | 2 | 0 | 1 | 1 | 1 |
| No Clear Factor | 1 | 11 | 31 | 3 | 1 | 0 | 8 | 3 | 1 |
| Failure to Yield | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 0 |
| Illegal or Unsafe Speed | 2 | 29 | 10 | 17 | 7 | 7 | 1 | 21 | 0 |
| Following Too Closely | 0 | 3 | 11 | 0 | 1 | 1 | 3 | 0 | 0 |
| Disregard Traffic Control Device | 0 | 0 | 0 | 0 | 0 | 0 | 1 | 0 | 0 |
| Improper Passing | 0 | 0 | 0 | 0 | 0 | 1 | 0 | 0 | 0 |
| Improper or Unsafe Lane Use | 0 | 2 | 2 | 2 | 0 | 0 | 0 | 0 | 0 |
| Improper Parking/Stopping | 0 | 2 | 0 | 0 | 0 | 0 | 0 | 0 | 0 |
| Over-Correcting | 0 | 1 | 0 | 0 | 2 | 0 | 0 | 0 | 0 |
| Inattentive/Distracted Driver | 1 | 2 | 7 | 0 | 2 | 2 | 3 | 3 | 0 |
| Driver Inexperience | 0 | 1 | 0 | 0 | 1 | 0 | 0 | 0 | 0 |
| Chemical Impairment | 0 | 1 | 1 | 0 | 1 | 0 | 0 | 3 | 0 |
| Other Human Contributing Factor | 0 | 2 | 2 | 0 | 1 | 0 | 1 | 0 | 0 |
| Vehicle Obscured | 0 | 0 | 0 | 0 | 1 | 0 | 0 | 0 | 0 |
| Defective Tire/Tire Failure | 0 | 0 | 1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Skidding | 0 | 1 | 1 | 0 | 0 | 1 | 0 | 4 | 0 |
| Other Vehicle Defects or Factors | 0 | 0 | 2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Weather | 0 | 4 | 1 | 1 | 3 | 2 | 0 | 6 | 0 |
| Other/Unknown | 0 | 0 | 2 | 0 | 0 | 0 | 1 | 2 | 0 |
| Total | 4 | 73 | 90 | 25 | 22 | 14 | 19 | 44 | 2 |

**Sample Calculation: Number of Accidents per Million Vehicles Crossing**

For a given bridge the number of accidents per million vehicles was found by dividing the number of accidents per year by the bridge's AADT. The values were then averaged for the number of years before and after an epoxy overlay was installed. This average is a small percentage and for the sake of presenting the data this percentage was reported in terms of millions of vehicles. The following is a sample calculation using data from Bridge 9066 [Poly-Carb, Flint] using data from Table 48.

Table 48: Data to Calculate Number of Crashes Normalized to AADT for Bridge 9066

| Year | Year Relative to Installation | Number of Accidents | Yearly Accidents per Yearly Traffic |
|---|---|---|---|
| 2003 | -7 | 9 | 7.95404E-07 |
| 2004 | -6 | 3 | 2.65135E-07 |
| 2005 | -5 | 11 | 9.72161E-07 |
| 2006 | -4 | 3 | 2.65135E-07 |
| 2007 | -3 | 9 | 7.95404E-07 |
| 2008 | -2 | 6 | 5.3027E-07 |
| 2009 | -1 | 12 | 1.06054E-06 |
| 2010 | 0 | 3 | 2.65135E-07 |
| 2011 | 1 | 6 | 5.3027E-07 |
| 2012 | 2 | 3 | 2.65135E-07 |
| 2013 | 3 | 8 | 7.07026E-07 |

Bridge AADT: 31000 vehicles/day

$$\text{Yearly Accidents per Yearly Traffic} = \frac{\#\ \text{of Accidents}}{\text{AADT} * 365}$$

For 2003: Yearly Accidents per Yearly Traffic $= \frac{9}{31000*365} = 7.95404\text{E} - 07$

To find the number of accidents per million vehicles before installation

$$\frac{\Sigma(\text{Yearly Accidents per Yearly Traffic})}{\text{Number of Years Before Installation}} = \frac{0.000495}{7} = 0.000071\% = 0.707 \frac{\text{Accidents}}{\text{Million Vehices}}$$

**Benefit-Cost Ratio Calculation**

A complete example of this calculation is provided below using data from Bridge 69006 as an example.

In order to calculate the Benefit-Cost Ratio the number of crashes based on their severity must be tallied for before and after the installation of an HFO system (Table 49).

Table 49: Number of Crashes per Year by Severity (Bridge 69006)

| Year Relative to Installation | Year | No Injury | Possible Injury | Non-Incapacitating Injury | Incapacitating Injury | Killed |
|---|---|---|---|---|---|---|
| -6 | 2003 | 5 | 0 | 1 | 0 | 0 |
| -5 | 2004 | 3 | 1 | 0 | 0 | 0 |
| -4 | 2005 | 5 | 2 | 0 | 1 | 0 |
| -3 | 2006 | 6 | 0 | 0 | 0 | 0 |
| -2 | 2007 | 4 | 0 | 0 | 0 | 0 |
| -1 | 2008 | 1 | 0 | 1 | 1 | 0 |
| 0 | 2009 | 1 | 0 | 0 | 0 | 0 |
| 0.5 | 2009 | 0 | 1 | 0 | 0 | 0 |
| 1 | 2010 | 1 | 1 | 0 | 0 | 0 |
| 2 | 2011 | 2 | 1 | 0 | 0 | 0 |
| 3 | 2012 | 1 | 1 | 0 | 0 | 0 |
| 4 | 2013 | 4 | 0 | 0 | 0 | 0 |
| Total Number of Crashes | Before | 25 | 4 | 2 | 2 | 0 |
| | After | 8 | 4 | 0 | 0 | 0 |

With the totals calculated (Table 49), the averages are found for each severity and multiplied by the associated cost of crashes based on the figures given in Table 5. The difference between the before and after associated costs is the average annual benefit (Table 50). This average annual benefit is multiplied by four in order to find the benefit over the average lifespan of an HFO system. The average four year benefit is divided by the estimated cost of an HFO installation (found in Table 51) to obtain the benefit-cost ratio.

Table 50: Benefit-Cost Ratio Calculation (Bridge 69006)

| Severity | Average Annual Number of Crashes | | Cost of Crash per Severity | Associated Cost of Crashes | |
|---|---|---|---|---|---|
| | Before | After | | Before | After |
| No Apparent Injury | 3.846 | 1.778 | $4,400 | $16,923.08 | $7,822.22 |
| Possible Injury | 0.462 | 0.889 | $30,000 | $13,846.15 | $26,666.67 |
| Non-Incapacitating Injury | 0.308 | 0 | $61,000 | $18,769.23 | $0 |
| Incapacitating Injury | 0.308 | 0 | $280,000 | $86,153.85 | $0 |
| Killed | 0 | 0 | $3,600,000 | $0 | $0 |
| | | | Total | $135,692.31 | $34,488.89 |
| | | | Average Annual Benefit (Difference) | $101,203.42 | |
| | | | Average Four Year Benefit | $404,813.68 | |
| | | | Estimated Cost of HFO Installation (From Table 51) | $35,857.92 | |
| | | | Benefit-Cost Ratio (B/C) | 11.29 | |

Table 51: Calculation for Estimated Cost of HFO System Installation

| Bridge | Estimated Cost of HFO Installation ($/ft$^2$) | Width (ft) | Length (ft) | Area (ft$^2$) | Total Estimated Cost of HFO Installation ($) |
|---|---|---|---|---|---|
| 6347 | $10.00 | 28 | 674 | 18,872 | $188,720.00 |
| 9066 | $5.00 | 55 | 1,300 | 71,500 | $357,500.00 |
| 9067 | $5.00 | 55 | 1,300 | 71,500 | $357,500.00 |
| 9823 | $3.68 | 42 | 225 | 9,450 | $34,776.00 |
| 14809 | $5.67 | 38 | 163 | 6,194 | $35,119.98 |
| 14810 | $5.67 | 38 | 163 | 6,194 | $35,119.98 |
| 69006 | $3.68 | 42 | 232 | 9,744 | $35,857.92 |
| 27019 | $5.50 | 40 | 260 | 10,400 | $57,200.00 |
| 86013 | $5.50 | 44 | 193 | 8,492 | $46,706.00 |

Notes:

(1) The Estimated Cost per square foot was provided by MnDOT for every bridge except 27019 and 86013, for these two bridges an average of $5.50/ft$^2$ was used based off of the other bridges.

(2) The bridge widths were estimated during site visits and with the use of satellite imagery.

(3) The bridge lengths were provided by MnDOT