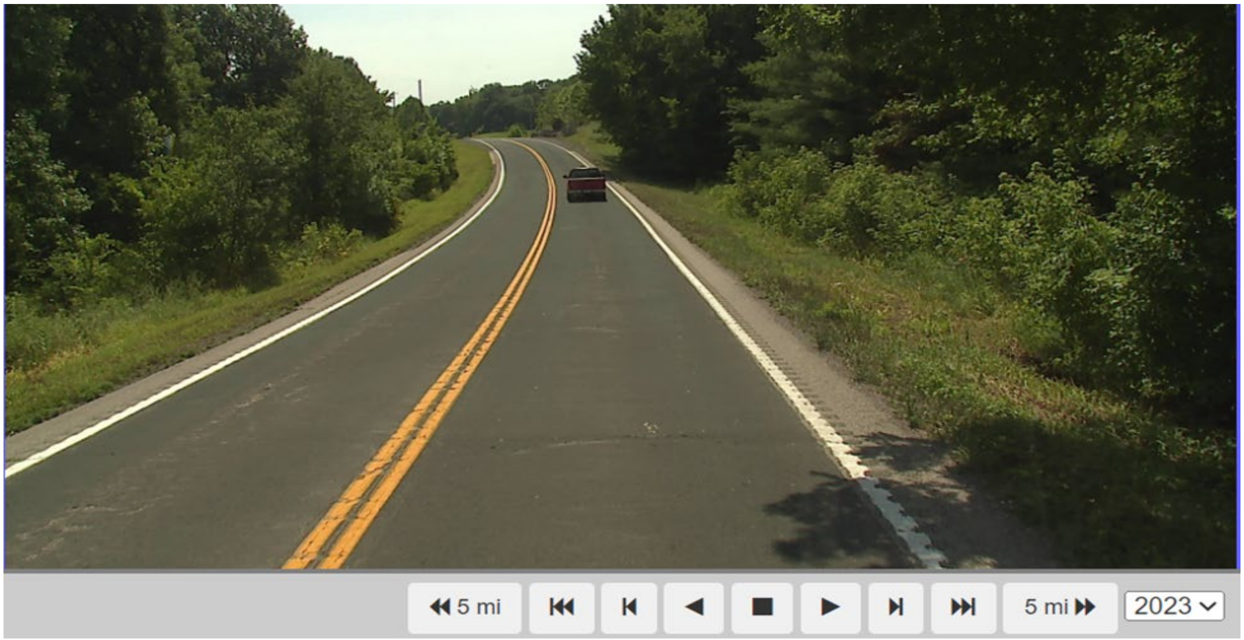
HFST Review



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16. Abstract

The Missouri Department of Transportation (MoDOT) has implemented High Friction Surface Treatment (HFST) in various locations across the state to reduce roadway departure crashes by improving pavement friction. To prevent premature failure and to ensure that the HFST performs adequately throughout the design life, this study evaluates the pavement distress and condition before and after the application of HFST. Along with the pavement distress and condition data, this study also evaluates the crash data of the HFST sections. Although analysis of distress data showed higher distress in some sections, it was the reflection of underlying distress, not the 'failure' of HFST, as observed from the ARAN images. Statistical analysis of the safety data revealed that HFST is around 66% effective in reducing crashes in wet conditions. Based on the evaluation of pavement condition and safety analysis, this study also provides guidance on the selection of future locations for the application of HFST.

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DISCLAIMER

The opinions, findings, and conclusions expressed in this document are those of the investigators. They are not necessarily those of the Missouri Department of Transportation, U.S. Department of Transportation, or Federal Highway Administration. This information does not constitute a standard or specification.

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The ARA project team recognizes and appreciates the services provided by MoDOT, including the condition and crash data of the high-friction surface treatment locations. The ARA team would also like to acknowledge and recognize the MoDOT Research team that provided guidance on completing the project activities.

EXECUTIVE SUMMARY

HFST is a widely accepted method to decrease roadway departure accidents by enhancing friction between the tire and pavement surface. MoDOT has implemented HFST in numerous locations throughout the state, as it presents a more cost-effective alternative to major roadway safety improvements like realignment and geometric correction in certain situations. Pavement conditions and crash data of HFST locations around Missouri before and after the application of the treatment were evaluated to create guidance or a set of recommendations for the successful implementation of HFST in new locations.

First, a systematic search of existing literature and a targeted email survey of 11 state or county representatives were conducted to identify common early distress signs in HFST, the mechanism of deterioration, and their impact on overall performance. The most common distresses observed in the HFST are delamination, cracking, raveling, and surface wrinkling. Aggregate loss or raveling is found to be more prominent if the surface is prepared through scarification milling, HFST installation at low temperatures, or both. Another form of failure of HFST is reported to be surface wrinkling, which is generally associated with higher traffic volume, loads, superelevation, and existing pavement strength. Calcined bauxite is widely used as aggregate in HFST, which provides high friction in the long run. Documenting distresses in the existing pavement prior to HFST installation is important to identify the root cause of early failure. Most of the surveyed state DOTs do not have established criteria or standards for selecting HFST sites based on the condition of the existing pavement. Only Florida and New Jersey have established criteria for selecting HFST sites based on existing pavement distresses such as cracking, rutting, stripping, and raveling.

After the literature review and focused survey, pavement distress data were assessed before and after HFST application to identify locations with early failure. A difference of three or more points in the average pavement condition (condition index) was observed for 32 locations in the years before the application of the HFST. Further examination showed that many of these sections were overlayed within a year of the HFST application. Based on this information, only the distresses and pavement conditions for the immediate year before the application of HFST were considered in the condition evaluation analysis for consistency. While some segments did display distress, it appears to be the underlying distress reflecting through. No "failure" of HFST was observed in the ARAN image.

Along with the distress, crash data of the HFST locations were also evaluated. Based on this analysis, 94 of 146 segments had a reduction in the number of crashes occurring after the application of HFST, with 12 segments having the same number of crashes following. It should also be noted that 11 segments had no crashes in the three years prior to application. Dividing the crash data by roadway conditions revealed that there were 925 wet crashes across 146 sections before the HFST application. This number was reduced to 166 after the application. However, only 48 of the 146 sections experienced decreased crashes post-construction, indicating that a large number of wet crashes were concentrated on a relatively small number of

sections with high crash numbers. A statistically significant difference in crashes was observed in wet conditions before and after the application of HFST, compared to dry conditions.

Guidance was proposed for the selection of new locations for HFST application using historical pavement conditions and crash data from HFST sites in Missouri. It was suggested that the condition of the existing pavement should be at least seven or higher for asphalt pavement and eight or higher for concrete pavement before HFST application, as per the PASER rating. Besides pavement condition rating, the guidance also proposed safety analysis with appropriate safety performance functions (SPFs). Historical data was also utilized to illustrate example calculations relevant to SPF calibration, site ranking, and the safety effectiveness of HFST on a two-lane, two-way rural undivided roadway segment with a horizontal curve. An average reduction of wet and dry crash frequency of approximately 66% and 7%, respectively, were observed after HFST application in such roadway segments. The proposed guidance on the HFST application is intended to be revised and updated based on feedback, suggestions, and engineering judgment from district and field engineers.

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1. INTRODUCTION

1.1. BACKGROUND

Safety is one of the primary concerns for the State Highway Agencies (SHAs), including the Missouri Department of Transportation (MoDOT). SHAs have implemented several innovative technologies to improve overall roadway safety, such as median guard cables, rumble strips, etc. Despite these improvements, there were 42,939 fatalities, with a staggering 2.49 million people being injured due to motor vehicle-related crashes in 2021 (Stewart, 2023). Horizontal curves have a higher risk of roadway departure crashes due to the high lateral friction demand compared to tangent (i.e., straight) roadways. Everyday Counts (EDC) initiative by Federal Highway Administration identifies High Friction Surface Treatment (HFST) as an innovative method to enhance roadway safety by increasing the frictional properties of the pavement surfaces in high crash rate areas such as horizontal curves, ramps, and intersections.

High Friction Surface Treatment (HFST) is an effective and efficient countermeasure to reduce roadway departure crashes by increasing the friction between tire and pavement surface (Bledsoe J. , 2015; Bledsoe & Lee, 2021; Anderson, et al., 2017; Wei, Xing, Li, Shan, & Guan, 2020). While HFSTs have shown an average life of 7 years to 10 years or more (Izeppi, Flintsch, & McGhee, 2010), many past studies have also reported early failures of HFSTs (Meggers, 2016; Wei, Wang, Tian, Li, & Shan, 2020). Early failure of HFST can occur for many reasons, such as poor condition of the existing pavement and poor construction practice. Despite their cost effectiveness in some locations, HFSTs are relatively expensive treatments with an average cost ranging between $25 to $50 per square yard (Federal Highway Administration, 2023). Thus, to ensure the HFST performs adequately throughout its intended design life, it is important to study the overall performance of HFSTs by considering the condition of the existing pavement before the application.

1.2. PROJECT OBJECTIVE

The objective of this study is to evaluate the performance of HFSTs by considering how the condition prior to application affects their overall performance. The specific objectives of this project are as follows:

- Develop a database of current HFST locations across Missouri and review their performance.
- Evaluate the effect of initial pavement conditions on HFST's performance.
- Provide guidance in determining the appropriate pavement condition suitable for the cost-effective application of HFST.

2. LITERATURE REVIEW

2.1. INTRODUCTION

A comprehensive literature search along with a survey of targeted Department of Transportations (DOTs) across the US was conducted to identify the common early distresses of HFSTs, their deterioration mechanism, and factors that affect their performances.

High friction surface treatment is applied as a safety enhancement technique for reducing roadway departure crashes by improving and restoring pavement friction. Typically, a “friction demand” exceeding the friction provided by the existing pavement surface due to the vehicle speed and roadway geometry causes many intersection and roadway departure crashes. HFST is generally constructed by bonding a high-quality, hard polish and abrasion-resistant aggregate to the pavement surface through a uniform and controlled application of polymeric resin binder or adhesive. This process increases the friction capacity of the pavement surface. The most commonly used aggregate for HFST is Calcined Bauxite. Polymeric resin and epoxy resin are commonly used as binders. Each DOT has different construction steps and requirements prior to the application of HFST, such as, approval of construction plans, contractor qualification, test strip construction, etc. However, the two most basic steps followed during the construction of every HFST site are surface preparation and application. A brief general discussion of these two steps is provided below:

2.2. SURFACE PREPARATION

Prior to the application of HFST, the existing pavement is checked for cracks, potholes, and other surface defects. Not addressing the existing surface defects adequately typically causes accelerated deterioration or premature failure of the applied HFST. Surface preparation involves the following steps:

- Any defects in the existing pavement, such as spalls, potholes, raveling, and rutting, should be repaired prior to the application of HFST. Areas with inadequately sealed joints should be cleaned and filled. Minor rutting and heaving may be allowed by the agency; however, HFST is not intended to be a repair method for such issues.
- Pavement cracks that are ¼ inch wide or over should be sealed with agency-approved sealant. The application of HFST is delayed in the event of an asphalt-based sealant being used. The wait period is typically 30 days (Wilson & Mukhopadhyay, 2016). Similarly, a wait period is also applicable to pavement surfaces with new asphalt or concrete surface layers.
- FDOT specifications suggest mill and fill of the top layer for the following conditions:
    - Cracking observed in the wheel path or outside of it exceeds six percent or more
    - Rutting of 0.25 inch or more
    - Bleeding of the surface

    - Raveling
- Removal of small oil spots may not be practical, but large oil spots should be removed through a mill and fill of the surface layer.
- Existing pavement markings should be covered, and utilities and other structures adjacent to the application area should be protected.
- The surface should be free from any dirt and debris through sweeping and high-pressure air wash.
- Oils, weak surface mortars, and curing compounds should be abrasively cleaned by shot blasting. The shot-blasted clean surface must meet the concrete surface profile rating specified by the agency. The surface should be swept and air-washed again after shot-blasting.
- The surface should be free from any moisture, and compressed air may be used to achieve a dry deck surface.
- For epoxy resin, the recommended surface temperature during HFST application is between 60°F-95°F. Surface temperature below or above the recommended temperature range generally leads to undesired effects. Along with temperature, the probability of rain, humidity, and wind is also considered during the application of HFST.

2.3. HFST APPLICATION

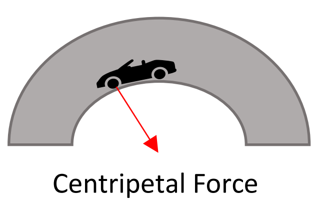
After preparing the surface, HFST is applied in four steps: 1) binder mixing, 2) binder application, 3) aggregate placement, 4) curing of binder, and sweeping of loose aggregates. The first three steps can be done using automated, semi-automated, or manual methods. A brief description of these steps is as follows:

- Binder mixing: Appropriate proportioning and adequate mixing is critical for binder preparation. For best results, it is recommended to use a vehicle-mounted mechanical system to meter, mix, and monitor the binder resin (Wilson & Mukhopadhyay, 2016). It is also important to ensure that it does not introduce any bubbles into the binder.
- Binder Application: Binder is applied uniformly to achieve a 50% embedment depth of the high-friction aggregate (Wilson & Mukhopadhyay, 2016).
- Aggregate Application: Aggregate should be spread uniformly to cover the whole binder area with no wet uncovered spots. After the application of aggregates, the surface should have a uniform color.
- Curing and sweeping: Based on the surface temperature and type of binder, it may take several hours for the binder to set. The loose aggregates present on the surface should be swept away. However, the aggregates should be checked to see if they have developed sufficient bond strength by pushing with a hand for possible movement before sweeping.

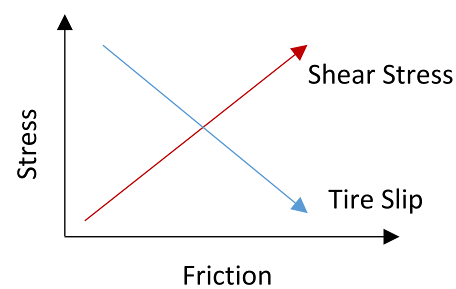
2.4. FAILURE MECHANISMS OF HFST

Past studies have mentioned several HFST failure mechanisms (Wei, Xing, Li, Shan, & Guan, 2020; Izeppi, Flintsch, & McGhee, 2010; Wilson & Mukhopadhyay, 2016; Yang, et al., 2019). They are:

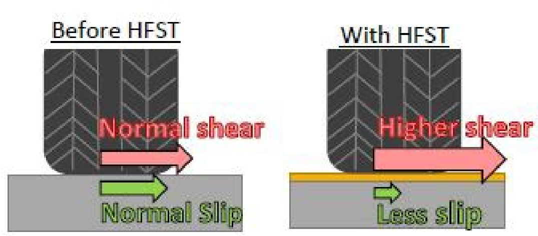
- Aggregate Loss or Raveling - The most common failure, which can occur both at the wheel path and non-wheel path locations. It occurs primarily due to insufficient aggregate embedment in the binder or due to a weak bond between the binder and aggregate.
- Reflective cracking – It is essential to prepare the surface of the existing pavement before installing the HFST. While the Epoxy or resin binder used in the HFST has some limited ability to resist the brittle cracking, traffic loads and freeze-thaw cycles will accelerate the propagation of the existing cracks through the HFST.
- Delamination – Failure of the bond between the HFST and existing pavement. Thermal incompatibility between the resin and substrate binder, moisture, and inadequate preparation of the binder are the primary reasons behind HFST failure by delamination.
- Uncured binder failure – The inability of the binder to adhere to the aggregate and/or the substrate. Poor workmanship and failure to follow appropriate specifications are the primary reasons for uncured binder failure
- Substrate failure – There are three mechanisms of substrate failure. The first is tearing. It occurs due to the centripetal force, which acts towards the center of the curvature, as shown in Figure 2-1(a) (Wei, Wang, Tian, Li, & Shan, 2020). Due to the braking and turning motion of the vehicle, this centripetal force generates shear stress between the tire and the pavement surface. A small amount of slipping also occurs at the same time between the tire and pavement surface. However, with high friction material, the slippage reduces, and the shear stress between the tire and pavement surface increases, leading to the tearing of the substrate. For example, Figure 2-1(b) shows that as the friction is increased due to the application of the high-friction surface material, the tire slip decreases, and the shear stress increases. Another illustration of increased shear stress and reduced slipping due to HFST application is shown in Figure 2-1(c). Substrate failure can occur due to this difference between the shear and slip stress**.**



(a) Direction of centripetal force



(b) Relation between shear stress and slipping



(c) Shear and slipping illustration

Figure 2-1: Qualitative mechanism of substrate tearing (Wilson & Mukhopadhyay, 2016)

The second form of substrate failure may arise from thermal incompatibility. Thermal incompatibility between HFST and pavement rises due to the differences in the Coefficient of Thermal Expansion (CTE). The typical CTE value for the HFST epoxy bauxite mortar is around 38.3×10^{-6} per °C. However, the value is around 12.6×10^{-6} per °C for HMA mixture (Wei, Xing, Li, Shan, & Guan, 2020; Wilson & Mukhopadhyay, 2016). This significant difference in CTE between HFST and HMA generates thermal shear stress at the interface between the two layers. The substrate will fail in tearing if the thermal stress value exceeds its fracture strength.

A third substrate failure mechanism can occur due to the entrapment of moisture. Moisture can be trapped between the HFST and existing pavement surface during construction, or it may infiltrate later during the service life. Regardless of the method, the entrapped moisture can cause substrate failure under traffic load and high temperature.

It is important to mention that delamination and substrate failure are two different failure mechanisms of HFST. Delamination occurs at the interface between the existing pavement surface layer and HFST, while substrate failure occurs in the surface layer of the existing pavement.

Many State Departments of Transportation (DOTs) have developed specifications for the construction of HFST. These specifications mostly focus on materials, surface preparation, and application. Calcined bauxite and polymer or epoxy resin are mainly used for the HFST. The specifications outline test methods and acceptable values for the materials to be used in the field. For example, the ultimate tensile strength of the resin binder should be between 2,000 psi to 5,000 psi.

Based on a review of the existing literature, very few DOTs have documented the early distress or the impact of the existing pavement condition on the performance of the HFST after installation.

The following subsections present case studies documented by different DOTs.

2.5. CASE STUDIES

2.5.1. Florida

Wilson and Mukhopadhyay studied the performance of the 23 HFST sites in Florida (Wilson & Mukhopadhyay, 2016). A few years after construction, 16 sites were in good or fair condition, while the remaining seven sites were in poor or worse condition, as shown in Figure 2-2.

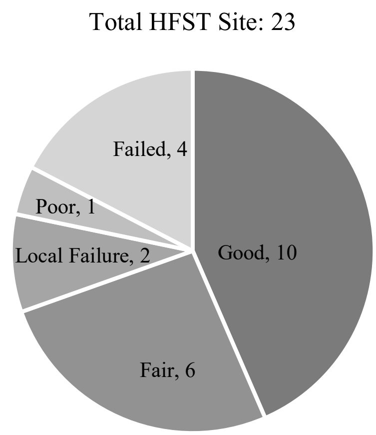


Figure 2-2: Condition of HFSTs in Florida (Wilson and Mukhopadhyay, 2016)

The three most prominent distresses observed were raveling, uncured binder failure, and delamination. Ten of the 23 sites studied by Wilson and Mukhopadhyay had raveling, four experienced uncured binder failure, and 2 had delamination. Apart from crash statistics, the exact pavement condition prior to the installation was not presented. However, Wilson and Mukhopadhyay discussed the performance and early distresses observed in the HFST. The performance of the HFST sites in Florida is summarized in Table 2-1.

Table 2-1: Performance and conditions of the selected HFST sites in Florida (Wilson & Mukhopadhyay, 2016)

Site	Existing Pavement Surface Construction Year	Type	HFST Applied in (Month-Year)	HFST Application Type	HFST Condition Reviewed in (Month-Year)	FN40	Distresses Observed	Experience
I-275 Off Ramp (Tampa)	2004	HMA Friction Course	Dec-2011	Single	N/A	Before: 30, two year after: 84, three year after: 64	Uncured Binder failure, aggregate loss, delamination, stripping of HMA substrate	The inexperienced contractor had issues with mixing the appropriate amount of epoxy, which resulted in spots with inadequate epoxy. The condition of the second half of the project after the HFST application was particularly bad, which required the HFST from the first application, along with the existing HMA surface, to be milled and replaced. In the second application, the aggregate was applied manually and with the mechanical truck. The first section, which was not corrected, experienced the majority of the distresses. It had uncured binder failures and potholes. Many HFST locations did not have any aggregates and showed delamination and stripping of the HMA substrate. Distresses were nonexistent in the second half, which may have been attributed to the correct application of HFST on an undamaged surface with lower traffic stress.
Off Ramp from SR 60 to I-75 NB (Brandon)	2004	HMA Friction Course	Dec-2011	Single	February 2015	Before: 34, two year after: 62	Depressions, raveling and potholing of substrate, delamination	The contractor was inexperienced and had issues with mixing the appropriate amount of epoxy. It resulted in several spots not being adequately set up for aggregate application. However, the portion of this project was reconstructed. HFST failures and potholes in the substrate were observed in the locations with inadequate epoxy. Depressions in the pavement, delamination from the substrate, and stripping and raveling of the substrate materials were also observed. Reconstruction of the project was scheduled and may have been completed by the time of writing this report.
Memorial Blvd I-4 Ramp Connection (Lakehead)	2009	HMA Friction Course	Oct-2011	Double	July 2014	After: 82	Raveling in few places	Overall, the project performed well.

Table 2-1: continued

Site	Existing Pavement Surface Construction Year	Type	HFST Applied in (Month-Year)	HFST Application Type	HFST Condition Reviewed in (Month-Year)	FN40	Distresses Observed	Experience
I-95 Mainline and Ramps	N/A	N/A	May-2014	N/A	less than a year	Before: 43, After 77	Few places with splotchy areas with poor epoxy	Overall, the project performed well, but there were some areas where the epoxy did not adequately set due to the improper mixing.
I-95 Off ramp to Congress (Boca Raton)	N/A	HMA Friction Course	Oct-2010	Single	2015	After:79, three Year later: 65, five Year later: 60	Transverse cracking, random cracking, and raveling of low to high severity	The site was L-shaped, with a stop signal. It can be divided into three regions based on the distresses observed. The approach area to the stop location had transverse cracks. Near the stop location, cracks were random, almost forming block cracks, and had high severity. At the turn, there was moderate to high raveling. Random cracking of low severity was observed at the end of the section. The pull-off test results varied between 36 and 145 psi, and all failed in asphalt. The pull-off test values, particularly the lower ones, suggests a poor condition of the substrate.
Miami Beach	2008	Dense Graded Friction Course	Aug-2011	N/A	2014	Before: 36, After: 69, one year later: 57	Random Cracking	Immediately after construction, the project had uncured binder failure in several random locations. Fixing those random locations created splotchy areas. The location had some random cracking. As the pavement was new, it was believed that the cracking was initiated due to thermal incompatibility between the HFST and the existing surface course.
I-595 Off-ramp to FLA	N/A	Combination of HMA friction course and concrete bridge decks	2011	N/A	2014	Before: 44, one year later: 88, two year later: 77, four year later: 69	Raveling, irregular Depressions	Inadequate application of the binder during HFST construction caused raveling. Irregular depressions are typically precursors to the potholes in the substrate, which may have been caused by poor mixing of the epoxy and stripping of the substrate due to the accumulation of moisture underneath the HFST.

2.5.2. Indiana

INDOT's HFST initiative included 21 projects. Wei et al. (Wei, Wang, Tian, Li, & Shan, 2020) conducted pre and post-construction inspections of these projects to determine the effects of Average Annual Daily Traffic (AADT), percent trucks, posted speed, drainage condition, and existing pavement condition (surface distresses) on the durability of the HFST. The majority of existing pavements in the 21 HFST sites were in good condition, some were in fair, and only one site, SR-257, was in poor condition. Prior to the installation, the condition of the existing pavements of six HFST sites was evaluated using FWD and GPR for structural adequacy, where the pavements were either in good or fair condition. Pavements with severely cracked areas were removed and patched with HMA. It was assumed that the epoxy for HFST would fill the transverse and longitudinal cracks; thus, no crack filling was applied. Only one site, SR-32, had a 1.5-inch surface milled and filled before installation. There was a 30-day waiting period for SR-32 before installing HFST. All the other sites had chip seals. Construction took place between August and October 2018. HFST sites were reviewed for early distress signs in April 2019, after an entire winter season, i.e., a freeze-thaw cycle. Early observed distresses were documented, and infrared images were used to identify the possible distresses underneath the pavement. A total of 14 cores from 4 sites were collected to observe the possible presence of distress. Pull-off tests were conducted at the laboratory to remove the site temperature effects. Adequate strength was observed at the interface between the HFST and HMA layer. All the samples tested for the pull-off test failed in the HMA layer.

The most common early distresses observed was reflective cracking. Ranking of the observed early distresses for the HFST sites around Indiana are as follows:

- Reflective cracking
- Substrate tearing or surface wrinkling
- Aggregate loss and delamination

There was no reflective cracking three months after the installation of HFST. However, reflective cracks started to initiate at a faster rate after three months. As expected, the more cracks in the existing pavements, the more reflective cracks are observed in the HFST. Thermal stress generated due to the CTE variations between the HMA surface and HFST also accelerated the propagation of the reflective cracks. Furthermore, the thermal infrared images showed that the epoxy was inadequate in filling the cracks in the existing pavement, as shown in Figure 2-3. Appropriately filling the cracks using crack fillers might have reduced the number of observed reflective cracks. A relatively lower number of reflective cracks was observed for SR-32, the site with a 1.5-inch surface layer milled and filled. It suggests that the surface treatment by milling and filling had slowed the progression rate of reflective cracking.

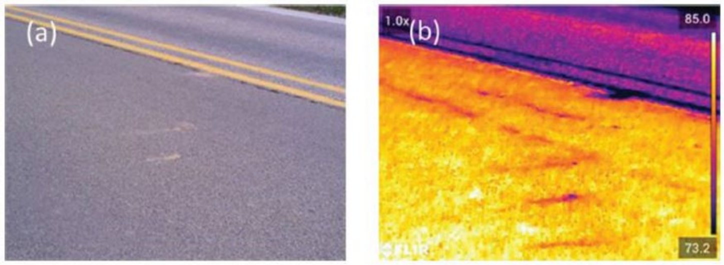


Figure 2-3: (a) Digital Image (b) Infrared Image of reflective cracks (Source: Wei et al., 2020)

Wei et al. (2020) noted the preparation of existing surfaces using scarification milling might have contributed to the observed aggregate loss and delamination of HFST. Scarification milling is a surface preparation technique that creates ridges and valleys on the existing pavement surface with a cutting depth of up to 0.75 inches. In Indiana, a rural two-lane highway may have up to 12% superelevation on the horizontal curve. At higher super elevation locations, the surface may be left un-scarified. Furthermore, the applied epoxy binder may flow to the lower side of super elevation during curing, as shown in Figure 2-4. This results in an uneven binder thickness, which can cause raveling.

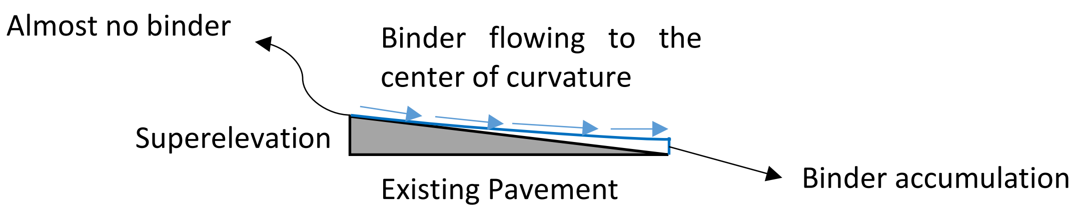


Figure 2-4: Flowing of the applied binders to the lower side of the super elevation during curing

Scarification milling also increases the interfacial shear stress between the HFST and the existing pavement surface (Wei et al., 2020). All the delaminated surfaces observed in Indiana had chip seals. Generally, shear stress increases with depth up to a certain point and then decreases with additional depth. This can be better represented with the schematic of shear stress acting on the cross-section of a beam, which is shown in Figure 2-5.

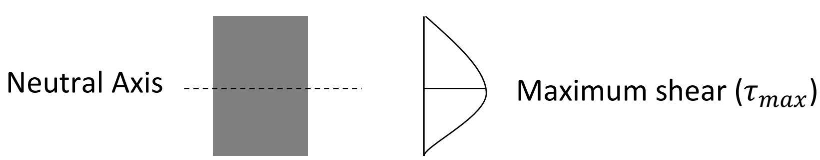


Figure 2-5: Schematic of shear stress distribution on the cross section of a beam

It was observed that the critical shear stress increases between 0.08 inch and 0.24 inch from the surface. The interface between the chip seal and the HMA layer is typically located at a greater depth than the critical shear location. However, as mentioned earlier, scarification milling takes out pavement surface up to 0.75 inches. It brings the interface between the chip seal and the HMA layer closer or into the critical shear stress zone, at least in the valleys of the sacrificed milling locations. Furthermore, the impact force of the rotating drums and cutters may also damage the bonding between the chip seal and the HMA layer of the existing pavement. If the underlying pavement has cracks, then scarification milling will help it propagate at a faster rate, ultimately leading to the delamination of the HFST layer. Typically, scarification milling provides better bonding between the old and newer surfaces in the case of micro surfacing, ultrathin-bonded wearing courses, and HMA overlays. However, it may not be an appropriate surface preparation technique for a chip sealed HMA pavements (Wei et al., 2020).

According to Wei et al. (2020) another reason for aggregate loss may have been the application of epoxy and curing it below 60°F with insufficient time for curing. The epoxy manufacturer recommended a 6-hour curing time if the epoxy is applied at 65°F. As the epoxy was applied at temperatures lower than 65°F, it is possible that the binder was not given enough time to cure before opening the road to traffic. Wilson and Mukhpadhhay do not recommend the application of epoxy binder below 60°F (Wilson & Mukhopadhyay, 2016).

Surface wrinkling or substrate tearing was also observed in 8 locations, particularly the locations with heavy and high volumes of traffic. High friction values, FN40 of more than 70, were observed for all the HFST sites after a year, regardless of the distresses.

A summary of the HFST sites in Indiana a year after installation is shown in Table 2-2, and a detailed summary of the HFST sites with multiple distresses are shown in Table 2-3.

Table 2-2: Summary of HFST sites in Indiana (Wei et al., 2020)

Pavement Rating before HFST Installation	Total Number of HFST Sites	Number of Sites with Distress after One Year	Percentage of Total HFST Sites with Distress after One Year
Fair	6	4	67
Good	14	7	50
Poor	1	1	100

Table 2-3: Condition of HFST sites in Indiana with multiple observed distresses (Wei et al., 2020)

Pavement	Existing Pavement Condition	Change in Crack Percentage from existing Condition	Aggregate Loss	Delamination	Surface Wrinkling	Comments
US 35a	Fair	-5	No	No	Yes	Vacuum Sweeping, High number of semi-trailers
SR 25	Good	-18	No	No	Yes	Vacuum Sweeping, Application below 60°F
SR 62a	Good	-14	No	No	Yes	Vacuum Sweeping, AADT greater than 2000
SR 62c	Good	+72	No	No	Yes	Vacuum Sweeping, AADT greater than 2000
US 24a	Good	-3	Yes	No	Yes	Scarification Milling, Application below 60°F, High number of semi-trailers
SR 23	Fair	-7	Yes	Yes	No	Scarification Milling, Application below 60°F
SR 65	Good	+78	No	No	Yes	Scarification Milling
SR 205 a	Fair	+23	Yes	Yes	Yes	Scarification Milling, Application below 60°F
SR 205 b	Fair	-5	Yes	Yes	Yes	Scarification Milling, Application below 60°F
SR 257	Poor	-45	Yes	No	No	Scarification Milling, Application below 60°F
SR 446	Good	-6	Yes	Yes	No	Scarification Milling, Application below 60°F
SR 450	Good	-37	No	Yes	No	Scarification Milling, Application below 60°F

2.5.3. Washington

Washington State DOT performed a preliminary study on the performance of HFST by installing it on a ramp with a horizontal curve (Anderson, et al., 2017). The total installation length was 500 ft with a width of 22 feet. HFST was applied only on the outer lane and shoulder of the two-lane ramp, which allowed the comparison of friction numbers (FN40) between the pavement with and without the application of HFST. HFST was applied twice: the first application was on August 30th, 2010, and the second application was in May 2011. The untreated portion of the roadway had a friction number of approximately 44. The treated area had a friction number of between 54 and 76 after the first and second applications of HFST, respectively. The performance of the treated location was observed for 5 years. The first survey was conducted 2 years after the installation, and the second one was 5 years after construction. Distress was observed in the first survey, and the severity continued to increase with time. Delamination in the wheel path was the most prominent distress, which is shown in Figure 2-6. Transverse cracking was also observed in some locations. Several causes, such as poor existing pavement conditions, shear stress from traffic, temperature-induced stress, and thick application of HFST, were suggested as the reasons for delamination. However, as the condition of the existing pavement was not documented prior to the installation of HFST, it was difficult to identify a single reason. Despite the distresses, the friction number (FN40) of the treated location was above 70 throughout the years.

(a) Condition after 2 years (b) Condition after 5 years

Figure 2-6: Condition of the HFST sites in Washington (Washington State Department of Transportation)

Variation of the friction numbers throughout the years for HFST sites in Washington is shown in Table 2-4. It shows that friction number remained higher regardless of the observed distresses.

Table 2-4: FN40 throughout the years (Washington State Department of Transportation)

Type	Year 0	Year 2	Year 5
Without HFST	44	46	45
HFST	76.7	72.8	73.9

2.5.4. Kansas

Four pavement locations in Kansas received HFST in 2009 (Meggers, 2016). Two on asphalt pavement and two on concrete pavement. Traffic volumes in those locations vary from 1,000 vehicles per day to 25,000 vehicles per day.

The K-99 and K-5 were asphalt pavements with 1.0 and 1.6 inches of existing HMA surface layers and AADTs of 1,550 and 1,000, respectively.

The I-35 and K-96 locations were concrete pavements. For these locations, the AADT was approximately 25,000 and 6,400, respectively. ASTM C881 Type III epoxy was used as a binder with flint aggregate. The aggregates were obtained from Picher, Oklahoma. Flint aggregates are generally used in Kansas for bridge deck overlay with polymers.

The performances and distresses observed in the four HFST sites in Kansas are summarized below:

- In 2012, HFST on location K-99 was covered with an asphalt overlay three years after construction. A total mill and fill was performed in 2013 due to the severe delamination of the HFST. Due to early failure, no distress data is available at this site. Skid resistance data showed that after one year of HFST application, the skid resistance values reduced from 8-15%.

- The asphalt pavement location, K5, was in relatively good condition four years after construction. However, reflective cracking from the existing pavement was observed, which might have been due to the inadequate filling of the cracks present on the existing pavement. The average skid resistance value after HFST application was 79.2, and after four years, the value reduced to 52.
- For the I-35 (PCCP), significant delamination was observed after four years. The location had reflective cracking as well. The average skid resistance value reduced from 64 to 44 in four years.
- The HFST was found to be in fair condition in K-96 (PCCP). The treated location was 866 feet long. The HFST was in good condition in the first half of the applied location. However, for the second half, delamination was observed similar to I-35. The average skid resistance reduced from 57 to 50 in four years.

2.5.5. Oklahoma

Oklahoma applied HFST at six locations in 2015 (Yang, et al., 2019). Three locations were on rural highways with lower traffic volumes, while the other three locations on I-40 and I-44 were selected to observe the performance of HFSTs under high volumes of traffic. The rural sites had a shorter radius of curvature (700-2,000 ft) and higher-grade percentages (-6% to 5%). The interstate locations had relatively larger radius of curvature and milder grade variations (-1% to -2.5%). The AADT on the rural site varied between 700 to 3,000, while the AADT on the interstate locations was between 80,000 to 120,000. Among the six HFST sites, five had calcined bauxite as aggregate, while one site (site no 1, rural highway) used a local mine chat aggregate. Site 1 and site 2 were very close to each other, with similar traffic and environmental conditions. No information was available regarding the condition of the existing pavement.

The performance of the HFSTs was observed for two years. The friction number (FN40) of the HFST sites in rural locations was reduced by 32% in two years, while the friction numbers in interstate locations were reduced by almost 41%. This is expected based on the higher volume of traffic on the interstate highway locations. Although crash reduction was observed for site 1 with local mine chat aggregate, it had a lower friction number compared to adjacent site 2 with bauxite aggregate. After two years in service, the average coefficient of friction in the wheel path of site 1 and site 2 were 0.59 and 0.8, respectively. The relation between friction number (FN40) and coefficient of friction (μ) are as follows:

$$FN40 = 100 \times \mu \tag{1}$$

The most common distresses observed at the HFST sites were patching, reflective cracking, raveling, and delamination. The three locations on the interstate highway experienced a higher amount of distress. On one of the HFST locations on I-40, reflective cracking was observed a few months after the construction, and delamination was observed after two years. Raveling was observed three months after the installation of the HFST on the I-44 locations.

2.5.6. Kentucky

Kentucky began using HFSTs in 2010, and by the fall 2012 around 160 sections were constructed or programmed for construction (Von Quintus & Mergenmeier, 2015). Although Kentucky selects the HFST sites based on crash analysis, before construction, they perform a field assessment based on several factors to determine the appropriateness of the treatment. Three years or more expected remaining service life of the existing pavement was one of the factors. Friction testing was conducted on 80 HFST locations in 2012. Distresses were observed in some locations, including the raveling of HFST aggregates, delamination of epoxy resin, and propagation of transverse cracks for existing pavement. No further information regarding materials, friction number, and distresses pre-and post-HFST application were provided (Von Quintus & Mergenmeier, 2015).

2.6. HFST SURVEY

As part of Task 1 of this project, an email survey was conducted to identify the effect of existing pavement conditions on the performance of HFST and overall safety measures. The survey was sent to 16 state transportation agencies. Chief of the office of the federal programs at Caltrans forwarded the email survey to all counties in the state of California. Among them, four counties responded. In total, 11 responses were obtained from county and state transportation representatives. Responses were obtained from the following states or counties: South Dakota, Wisconsin, Tennessee, Florida, Ohio, Illinois, Alaska, the City of Modesto, CA, Nevada County, CA, Pittsburg County, CA, and Tulare County, CA.

A summary of the aggregated responses to the survey questions is provided below:

2.6.1. Number of HFST sites

Figure 2-7 shows the number of HFST sites installed in different states and counties. The respondent from Ohio was not sure about the number of HFST sites, and the Tennessee respondent did not report an exact number but stated that multiple HFST sites have been installed since 2006.

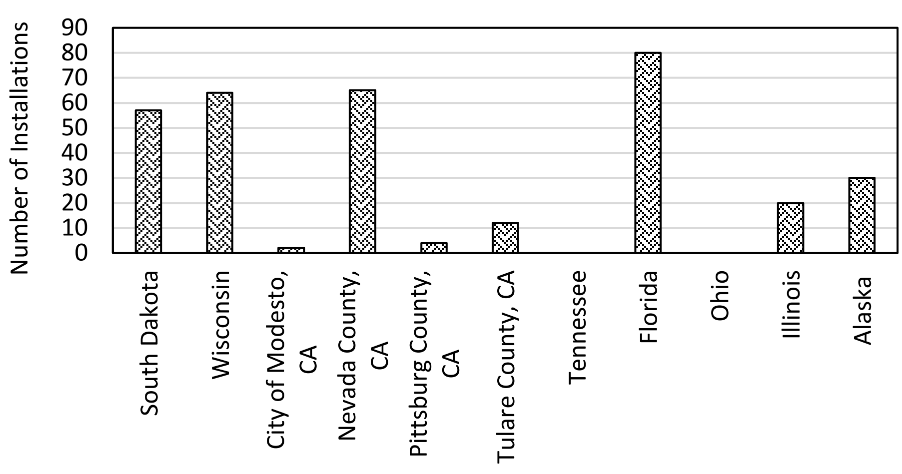


Figure 2-7: Number of HFST installations

2.6.2. Critical Factors for Site Selection

Figure 2-8 shows that all the respondents agree that crash data is the most important factor for HFST site selection, which is expected as it is a safety measure. After crash data, 54.5% of respondents reported road geometry, and 36.4% reported surface condition and age to be critical factors for site selection.

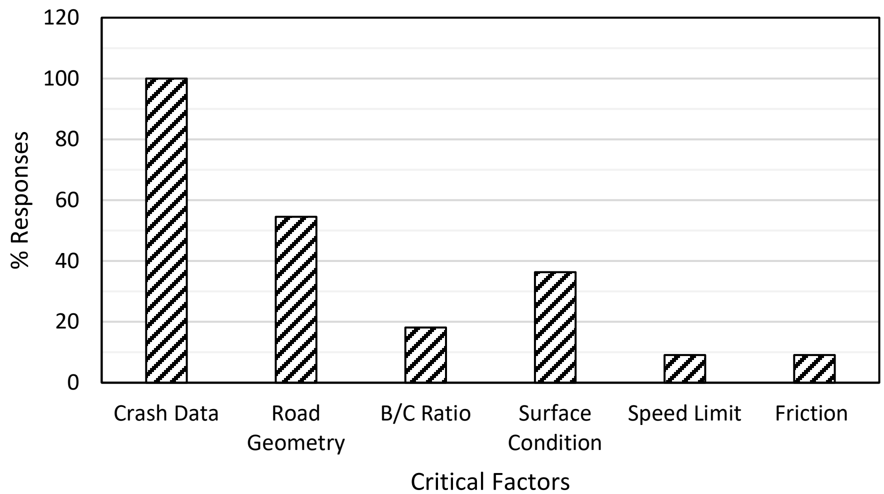


Figure 2-8: Critical factors for site selection

2.6.3. Traffic Factors

From the literature search, it was found that in Oklahoma, the friction number of HFST sites varied based on the traffic levels two years after the installation. Thus, a question on traffic level was included to identify if any of the states have a limit on the traffic level. Figure 2-9 shows that most of the respondents, around 55%, said they do not have any traffic limitations. Around 27% of the respondents said they have a traffic volume limit. For example, the traffic volume of the HFST sites in Illinois varies between 5,000-55,700 AADT by location. For around 20% of respondents, the speed limit plays an important role. For example, all the HFST sites in Florida had a speed limit lower than 45 mph regardless of the traffic volume.

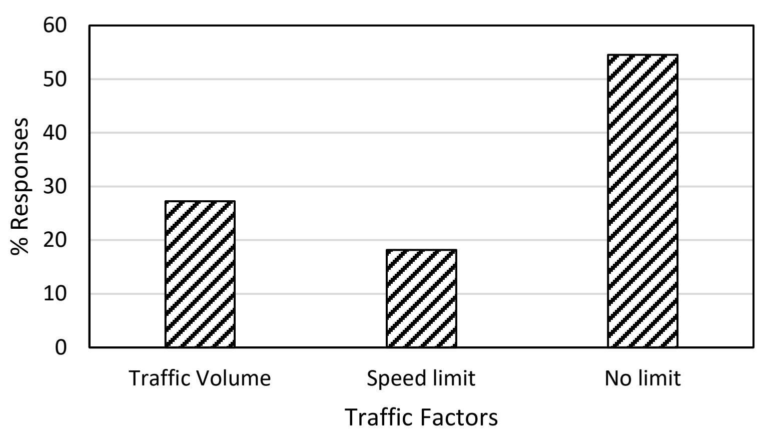


Figure 2-9: Traffic factors considered during the HFST site selection

2.6.4. Primary Aggregate

Figure 2-10 shows that 100% of the respondents answered that calcined bauxite was the primary aggregate for HFST. A representative from Tennessee answered that besides calcined bauxite, alternate aggregates are only used for bridge overlay. However, no further information was provided on the type of these alternate aggregates.

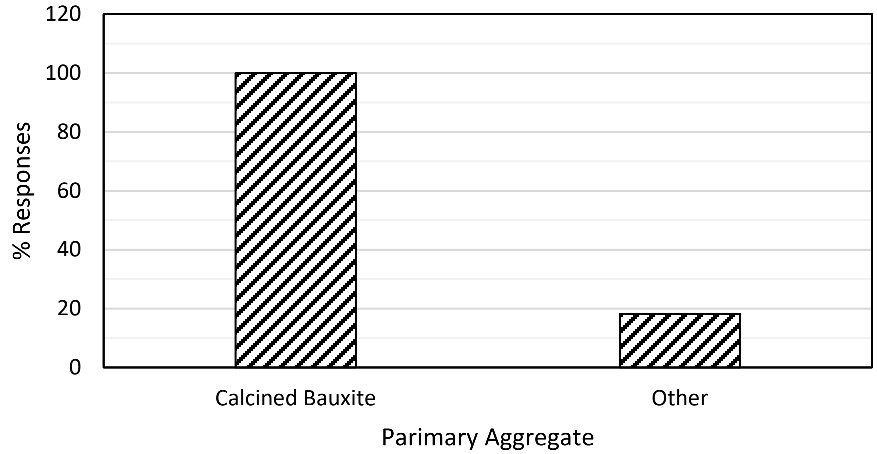


Figure 2-10: Primary aggregate in HFST

2.6.5. Primary Binder

Figure 2-11 shows that for more than 80% of the respondents, the primary binder for HFST is a polymer resin or epoxy binder. For around 20% of respondents, there is no preference as long as the binder specifications are met.

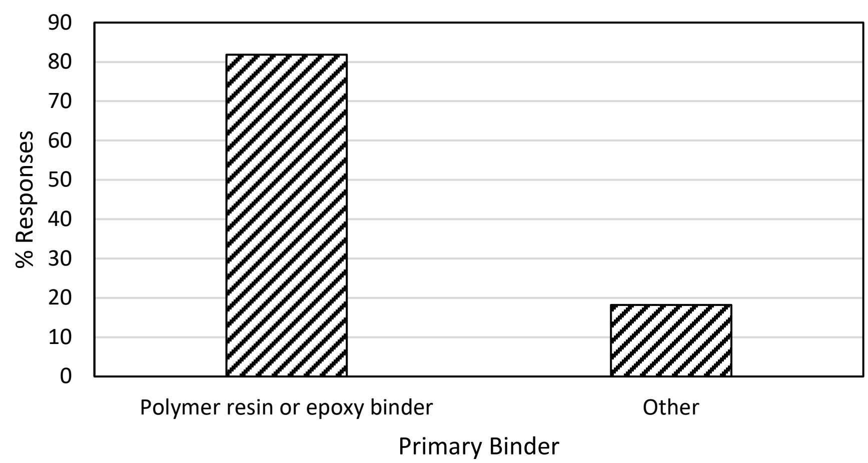


Figure 2-11: Primary binder in HFST

2.6.6. QA Test on Materials

From the obtained responses, it was observed that 54.5% of the states and counties have Quality Assurance test protocols for HFST materials, as shown in Figure 2-12. Around forty-five percent of the respondents reported no QA test of HFST materials. The four respondents who reported not having a QA test of HFST materials are three counties in California and the state of Ohio. It is important to note that Caltrans has protocols for QA tests on HFST materials in its specification.

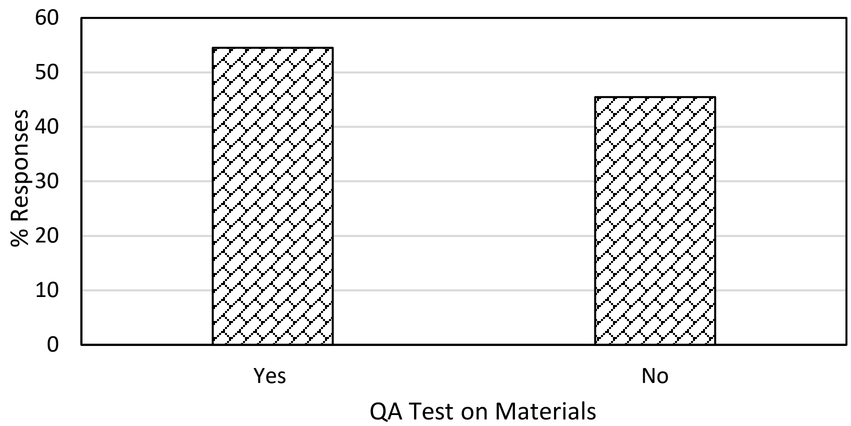


Figure 2-12: QA test of HFST materials

2.6.7. Exclusion based on Surface Condition

Around 80% of the respondents mentioned the exclusion of the proposed HFST site due to the condition of the existing pavement and age. Nevada County, CA, does not exclude any site. They perform repairs if needed. The City of Modesto, CA, did not provide any response to the survey question.

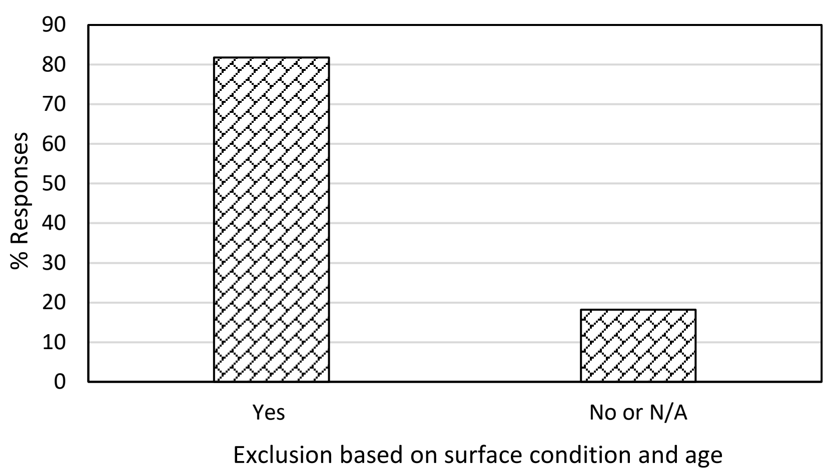


Figure 2-13: Exclusion based on surface condition

2.6.8. Minimum Standard for Distresses

Regarding the availability of a minimum standard for distresses, 54.5% responded with having no standard while 45.5% responded positively, as shown in Figure 2-14. Those who responded having minimum standard for distresses are Florida, Illinois, Tennessee, and Pittsburg County, CA. Illinois and Pittsburg County, CA did not provide any additional information on the standard apart from responding positively. Tennessee reported that the minimum standard for distresses is based on the visual inspection and time to next resurfacing or reconstruction.

If any of the following criteria are met for a particular site, Florida DOT mills and inlays the top layer of the flexible pavement before application of the HFST:

- Asphalt pavements with 6+ percent of cracking in or outside the wheel paths
- Widespread rutting > 0.25 inch deep
- Raveling surface
- Areas where layer debonding or subsurface stripping is suspected (verify with coring and other pavement forensics)
- Concrete single slab with moderate or severe distress, patching, or shattered in more than 3 pieces

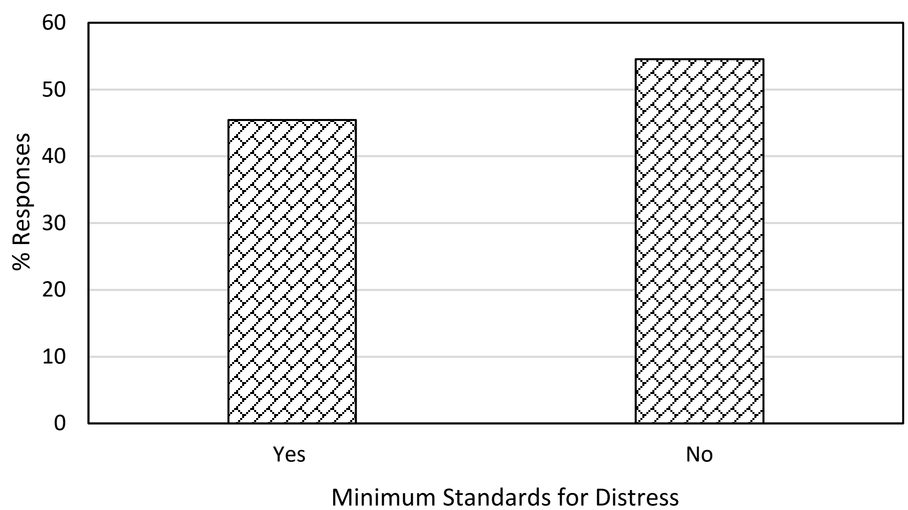


Figure 2-14: Minimum standard for distresses for HFST installation

2.6.9. Distress Data post HFST Installation

Figure 2-15 shows that none of the respondents collect distress data specifically for HFST sites after installation to monitor its long-term performance. About forty-five percent of respondents reported that distress data for HFST is collected as a part of the statewide annual distress data collection.

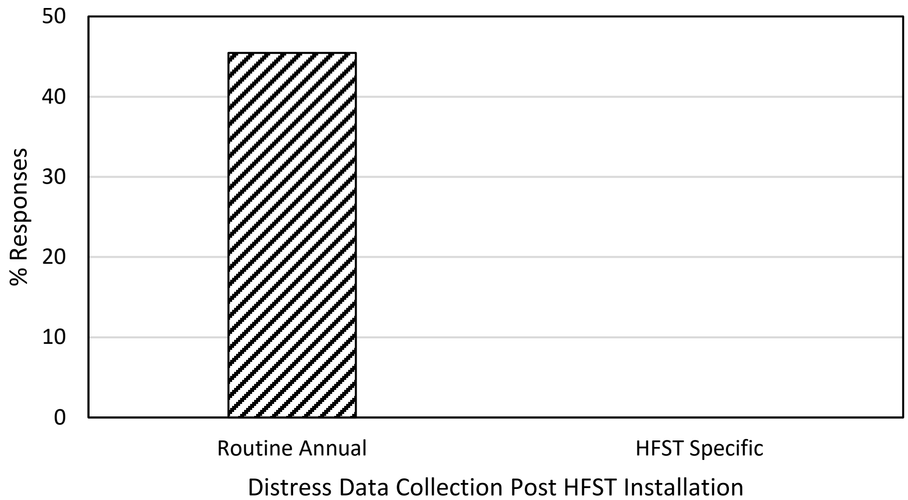


Figure 2-15: Distress data collection post HFST installation

2.6.10. Early Failures of HFST and Mechanism

The responses for early failures of HFST are shown in Figure 2-16. About 54.5 respondents observed early failures of HFST. The reasons or the common mechanism of the failure varies between delamination, raveling, cracking, and rutting.

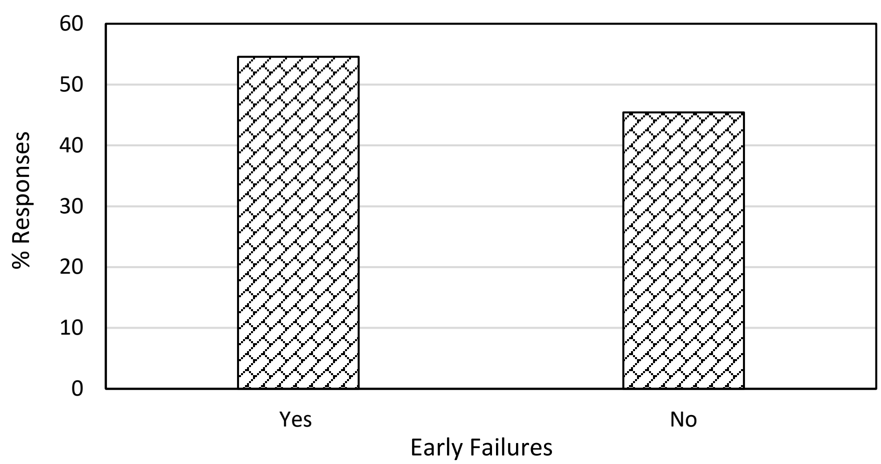


Figure 2-16: Early failures of HFST

Figure 2-17 shows delamination to be the most prominent mode of failure. More than 70% of the respondents identified delamination as the cause of HFST failure in their jurisdiction. The second most prominent mode of HFST failure was identified as raveling.

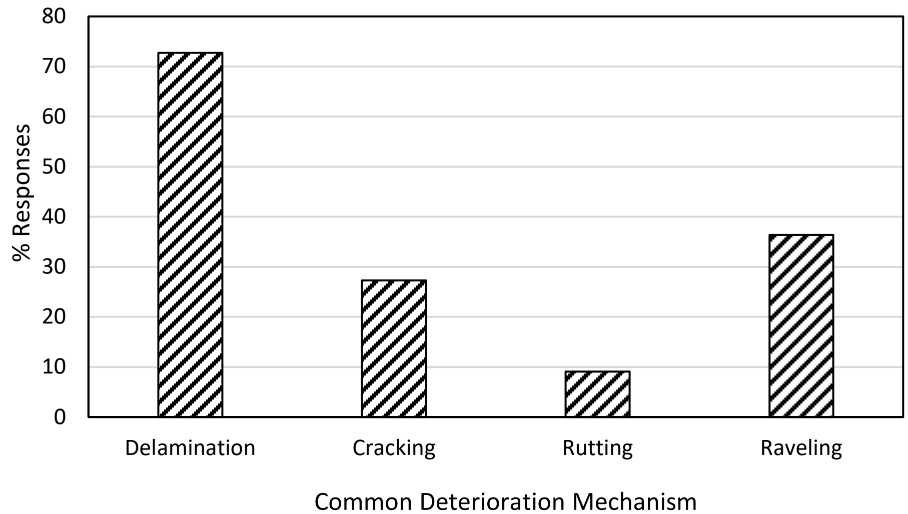


Figure 2-17: Deterioration mechanism of HFST

2.6.11. Prior Issues for Deterioration

Regarding the survey question on whether any prior issue has led to the failure of the HFST, around 45% responded positively, around 36% responded negatively, and around 18% did not have any response. The survey responses are shown in Figure 2-18.

Prior issues sometimes lead to the failure of the HFST. Based on the 40% of respondents who answered positively, these prior issues are inadequate surface preparation, pavement age, presence of moisture, and heavy traffic.

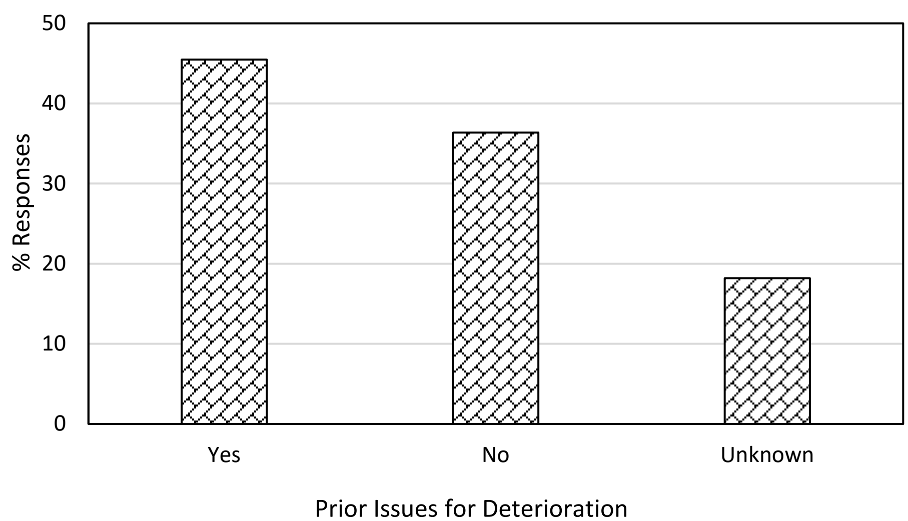


Figure 2-18: Prior issues for HFST deterioration

2.6.12. Performance of Deteriorated HFST

Figure 2-19 shows the responses obtained for the survey question regarding the effectiveness of the deteriorated HFST. Among the respondents, around 63% responded that deteriorated HFST still maintained adequate friction and reduced crashes. However, around 18% responded that post-HFST crash numbers did not reduce significantly from the pre-HFST crash numbers or that the reduction in crash numbers is unrelated to HFST because of early failures. More time is needed for 18% of respondents to determine performance as their HFST installations are relatively new.

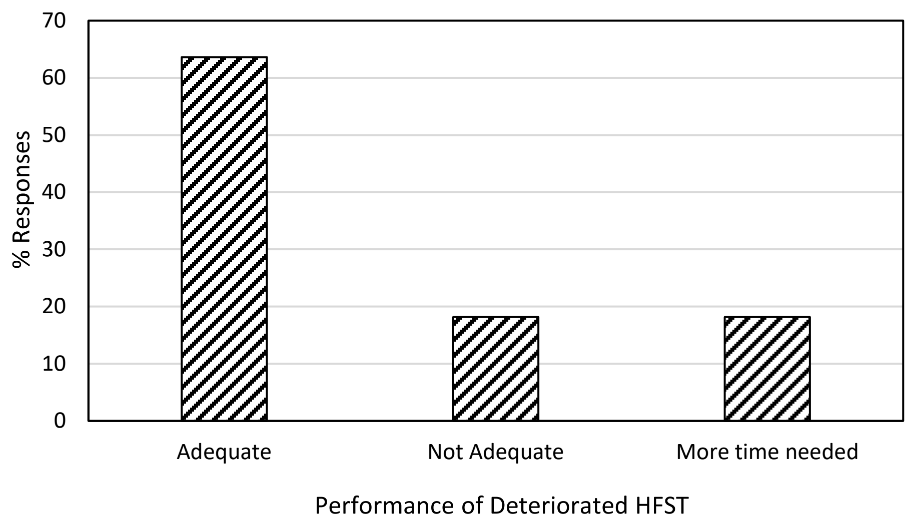


Figure 2-19: Performance of deteriorated HFST

3. DATA ANALYSIS

3.1. INTRODUCTION

The analysis performed in this report was conducted based on data provided in an Excel file received from MoDOT. The file contained information on pavement distresses, including rutting, cracking, the International Roughness Index (IRI), faulting, and average condition. It also contained general information on the HFST sections, including location, application date of HFST, travel direction, functional class, AADT, and the start and end log mile of the HFST sites. The data set included information for 295 HFST locations.

In addition to the pavement condition data, detailed crash data prior to the application of HFST were collected along with all available crash data for respective sections following the application. Several challenges were discovered when examining the historical records. The first challenge was that many locations did not have any crash records. It could be possible that no crashes were recorded in some sections, but there was a clear gap in the data. Further examination showed that for undivided travelways, all crash data is stored in the primary travelway direction, either South or East. For example, crashes occurring northbound on an undivided roadway have those crashes stored in the south direction. Similarly, crashes on an undivided roadway with a travel direction of the west will be stored in the east direction. Therefore, sections on an undivided roadway with a direction of North or West were removed from the analysis.

In addition to the crash record challenges, there were several locations with a travelway designation of “RP” and “CST” corresponding to ramps and city streets, respectively. While crashes are tied to ramps, there is no condition rating performed on ramps, nor is there video recorded. The CST segment is presumed to be a national highway system (NHS) route, not on the state system, and therefore contains no condition data.

After eliminating duplicate sections and the sections with no condition rating data, 165 distinct HFST segments were left with sufficient data for further consideration in this study.

It should be noted that 57 segments were placed in 2020 or later and have at most one year of condition data available and limited crash records. Twelve sites had multiple HFSTs applied, and seven sections did not have any post-HFST application pavement condition data. Eliminating these 19 locations brought the number of HFST sections available for analysis to 146. Table 3-1 provides a description of the change log on data cleaning.

Table 3-1: Data cleaning log

Available HFST Section Number	Description
295	Initial Data
262	Data with 0.1-mile pavement condition
167	Filtered undivided" and (travel direction West or North)"
165	Removed section 120 and 291, No HFST from ARAN or Ramp
158	Removed New application, no post application distress data: 25, 26, 39, 124, 194, 195, 204
146	Removed section: 3, 16, 74, 80, 85, 86, 131, 133, 134, 147, 257, 262 due to multiple applications

3.2. PRELIMINARY ANALYSIS

A preliminary check of the data showed a high variance in pavement conditions on several sections before the application of HFST. A difference of three or more points in in the average pavement condition (condition index) was observed for 32 locations in the years before the application of the HFST. Further examination showed that many of these sections were overlayed within a year of the HFST application. Based on this information, only the distress and pavement conditions for the immediate year before the application of HFST were considered in the subsequent analysis for consistency.

One of the major complications in determining an appropriate (or maximum) level of allowable distress was the lack of variety observed in the condition data. Of the 146 sections available for study, 30 were placed in conjunction with a resurfacing project, leaving only 116 sections where distress might be an issue.

Based on the current MoDOT rating methods, of the 146 sections available for study, 138 rated "good" based on Tracker Condition prior to application using the following criteria:

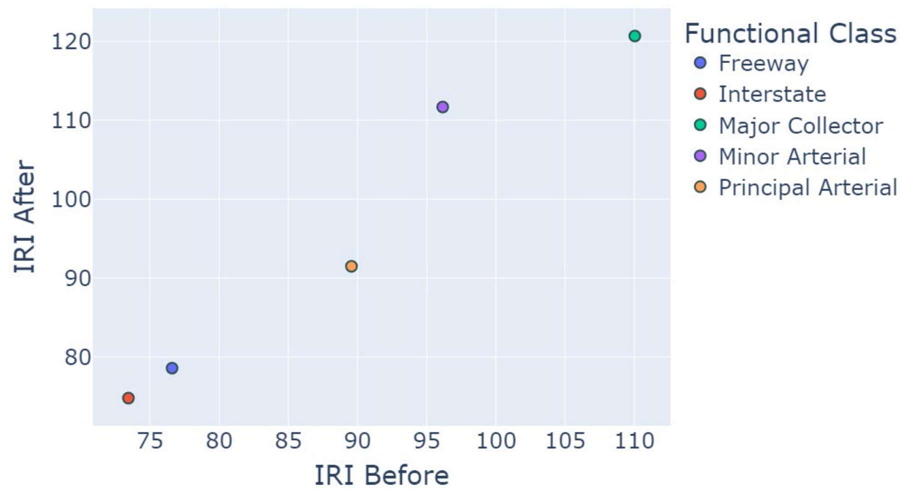
MAJOR: Average IRI < 100 OR (Speed Limit < 55 and Condition Index >=7)
MINOR: Average IRI <140 OR (Average IRI <170 and Condition Index >=6)
LOW VOL: Average IRI <170 OR (Average IRI <220 and Condition Index >=6)

A review of the video seems to confirm the data. Very few sections show major distress after application. Conversations with MoDOT staff indicated that many of the sections that came to mind as failures were on ramps, where no data was available, and that were eliminated from the dataset.

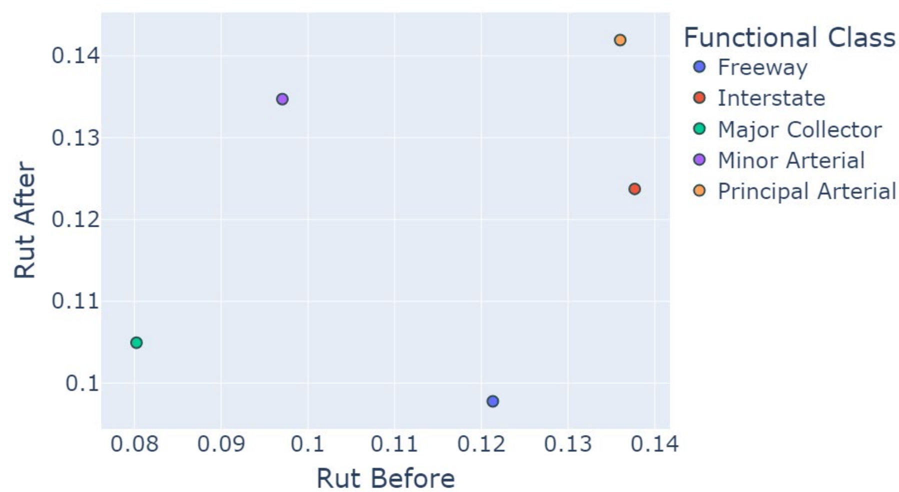
While some segments did display distress, it appears to be the underlying distress reflecting through, not the failure of the HFST itself. Again, this would be expected.

3.3. PAVEMENT CONDITION

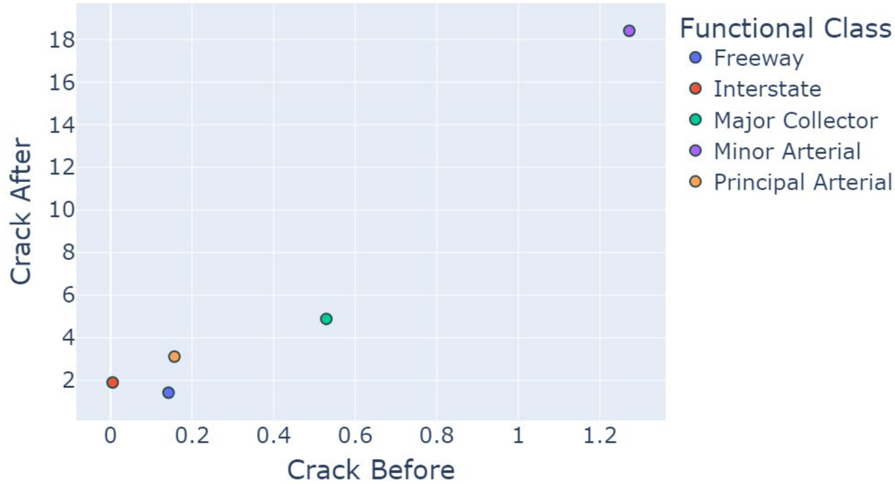
Although there are 146 HFST sections around Missouri, not all of them were constructed at the same time. Thus, the duration or length of post-HFST application pavement condition data available for sections differ. Some sections have pavement condition data available for one year, while others have up to eight years after the application of HFST. Figure 3-1 shows the variations in average distress three years after the application of HFST by functional class. Figure 3-1 (a) and Figure 3-1 (b) show that the average IRI and rutting did not increase significantly three years after the application of HFST. However, a large difference in average cracking was observed before and after the application of HFST, particularly in lower functional classes of pavements, such as arterials and collectors.



(a) IRI variations



(b) Rut variations



(c) Crack variations

Figure 3-1: Variations in average pavement condition three years after application of HFST

As variations in average cracking were observed for different functional classes of pavements after the application of HFST, it is also important to identify how this observation varies with pavement surface type. Figure 3-2 shows the variations in average cracking after the application of HFST based on both functional class and surface type. It shows that the highest amount of average cracking three years after application of HFST was observed for minor arterials with bituminous material (BM) and superpave (SP) pavement type. Moreover, at least 5% average cracking was observed in the interstate highway with Portland cement non-reinforced (PCN) and major collector with BM surface type there years after application of HFST.

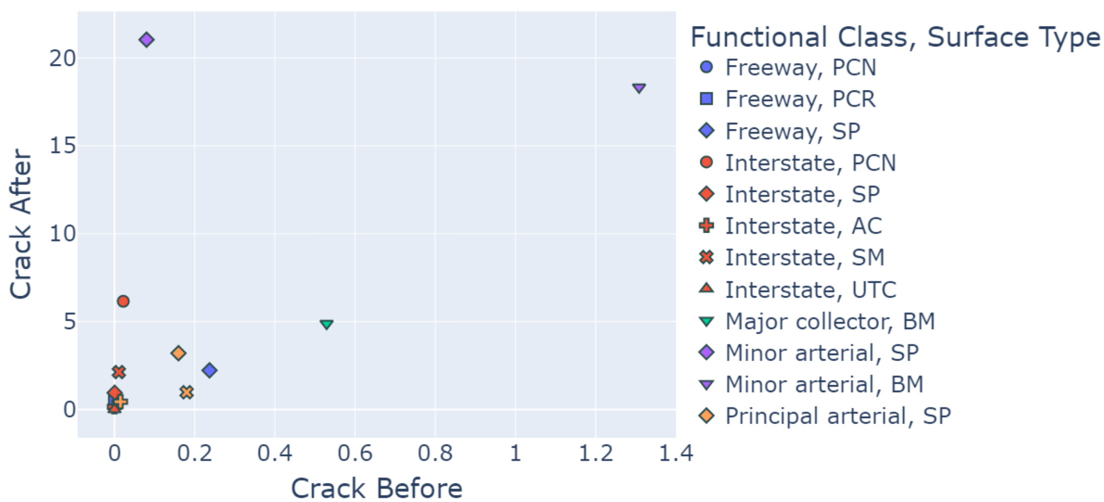


Figure 3-2: Observed variations in cracking based on pavement functional class and surface type

Despite these observed variations in average cracking by pavement functional class and surface types, failure of HFST could not be clearly recognized or identified from the ARAN images. Figure 3-3 shows the ARAN images of pavement surfaces before and after the application of HFST for various functional classes and surface types. As mentioned earlier, Figure 3-2 shows that interstates with PCN have more than 5% of average cracking three years after the application of HFST. HFST section 10 is located on an interstate with PCN surface type. Three years after application of HFST, average cracking for this particular section was reported to be around 16%. ARAN image of section 10 before and after application of HFST is shown in Figure 3-3 and Figure 3-4. Compared to Figure 3-3, Figure 3-4 shows apparent white patches or polishing effects on the surface but no “failures of HFST”, such as delamination, substrate tearing, etc. HFST sections 200 and 238 are minor arterials with BM and SP surface types, respectively. Pavement surface images before and after the application of HFST for these two sections are shown in Figure 3-5 through Figure 3-8. Similar to interstate highways, even with a high percentage of reported cracks three years after the application of HFST, these sections also did not show any “failure” of HFST.

Figure 3-3: ARAN image of section 10 before HFST application (source: MoDOT)

Figure 3-4: ARAN image of section 10 after HFST application (source: MoDOT)

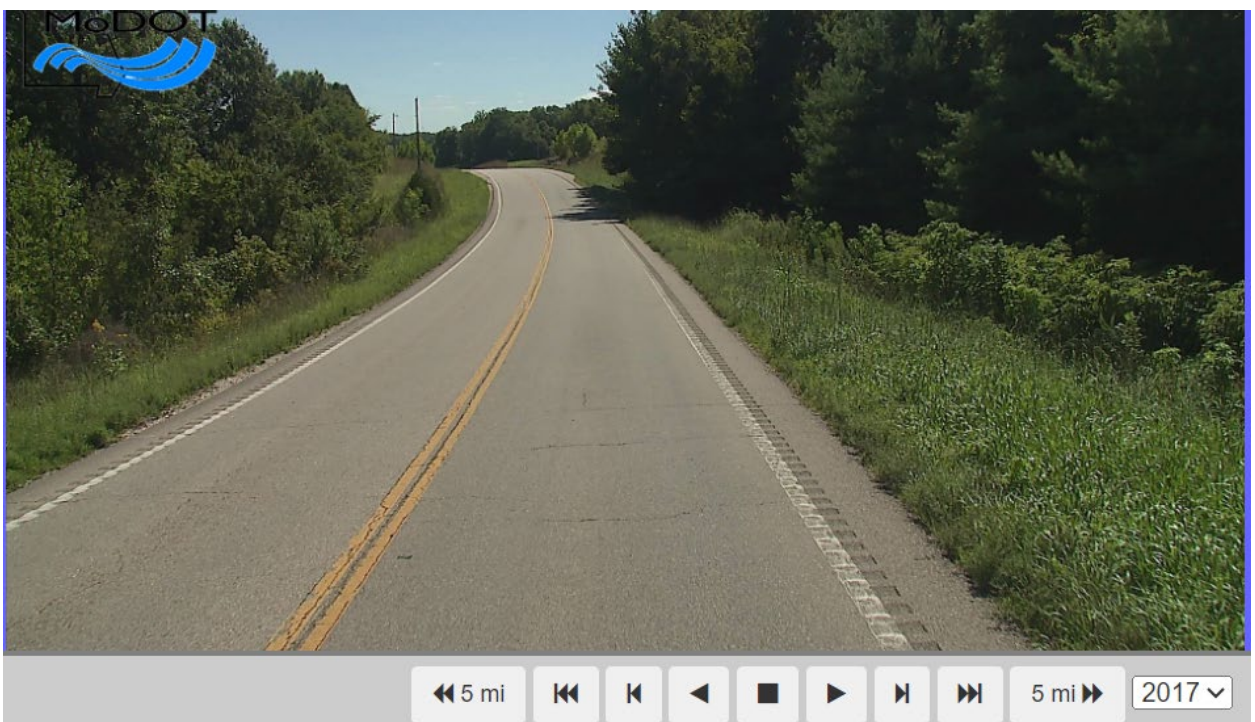


Figure 3-5: ARAN image of section 200 before HFST application (source: MoDOT)

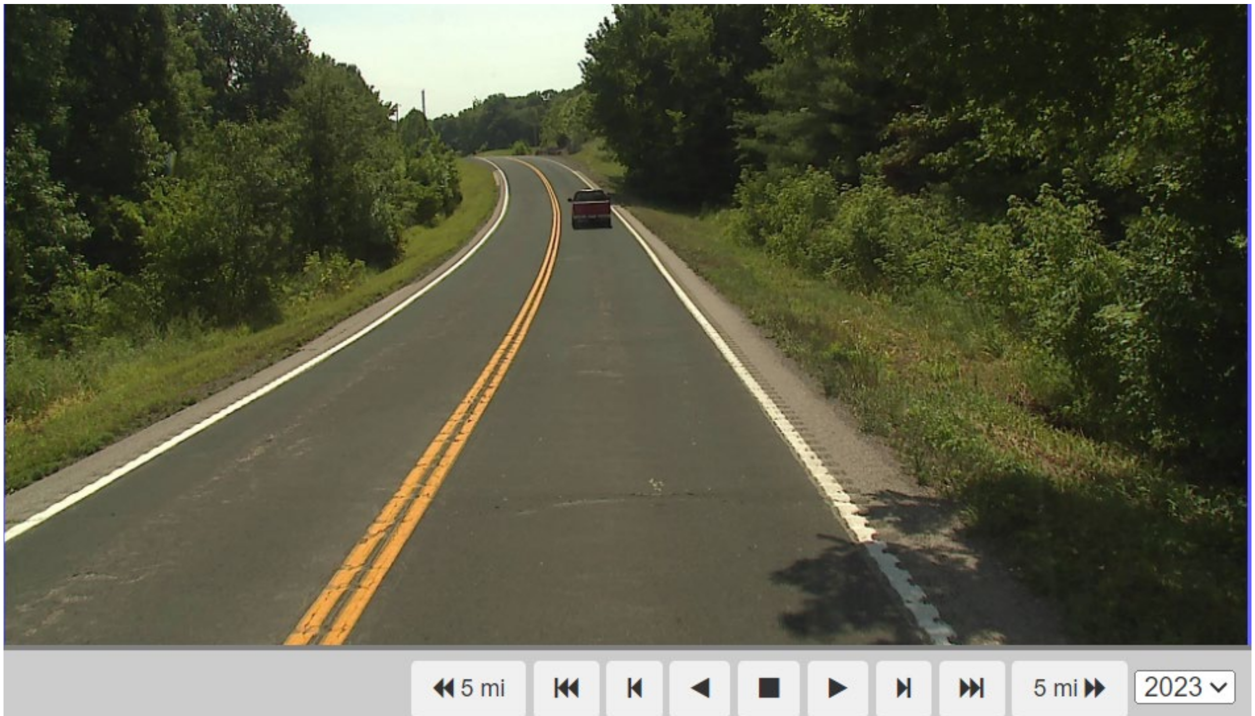


Figure 3-6: ARAN image of section 200 after HFST application (source: MoDOT)

Figure 3-7: ARAN image of section 238 before HFST application (source: MoDOT)

Figure 3-8: ARAN image of section 238 after HFST application (source: MoDOT)

As a side note, a conversation with MoDOT staff indicates that the average time between overlays on both Major and Minor roads now averages between eight and ten years. This study determined that the average pavement age at the time of HFST application was 8.8 years, indicating that, on average, the treatment is being applied near the end of its life cycle.

At this point in the study, it was decided to try to determine what “failure” would look like. Is the surface itself failing? Is there delamination from the underlying pavement, etc.? Or, has the underlying distress reflected through, but the HFST still providing the intended safety benefit? As it was not possible to identify a significant number of locations with premature or early failure of HFST from the ARAN images based on the pavement distress data, the research team decided to perform a safety analysis and use the PASER manual for recommendations regarding the guidelines on HFST application. An evaluation of the crash data was conducted to evaluate the safety benefit, which will be discussed in the next section.

3.4. CRASH STATISTICS

The initial effort looked for a positive benefit/cost relationship. To do this, crashes 3-years before and 3-years following the application were assigned monetary values based on the latest Federal cost assumptions by crash severity. This resulted in unexpected results. Of the 146 segments, only 78 had costs lower after application than before. This seemed inconsistent with logic and previous investigations. A sampling of records indicated that while the number of crashes might have been reduced, the severity of one or two crashes could drastically change the benefit-cost ratio. It is also important to note that crashes are rare events. The benefits of a crash reduction method are typically accrued over the average reduction in crash numbers at the network level, not for individual observations.

It was then decided to look at the crash reduction alone. Based on this analysis, 94 of 148 segments had a reduction in the number of crashes occurring after the application of HFST, with 12 segments having the same number of crashes following. It should also be noted that 11 segments had no crashes in the 3-years prior to application.

To perform the detailed crash data analysis, two crash data files were received from MoDOT. The first file had crash data three years before the application of HFST and three years after the application of HFST. The file divided the crashes into the following types:

- Fatal
- Serious injury
- Disabling injury
- Minor injury
- Property damage only

These different types of crashes were combined to obtain the total crash numbers in a particular HFST section. Along with the crash information, the file also contained other relevant information such as HFST section, length, functional classes of pavements, surface type, area designation, county, etc. Thus, observed crashes for different pavement functional classes, surface types, etc., were grouped to determine the weighted average crash numbers. The weighted average crash numbers were determined based on the segment length of each HFST section and respective AADT. From the difference in weighted average crashes before and after the application of HFST, the weighted average reduction in crash numbers was determined.

Figure 3-9 shows that the highest number of reductions in crashes after the application of HFST, around 11, was observed for the interstate class of pavements. A reduction of 3.5 and 2.9 crashes after the application of HFST was observed for the minor arterial and principal arterial classes of pavements, respectively. For other classes of pavements, a weighted average crash reduction of about two was observed after the application of HFST.

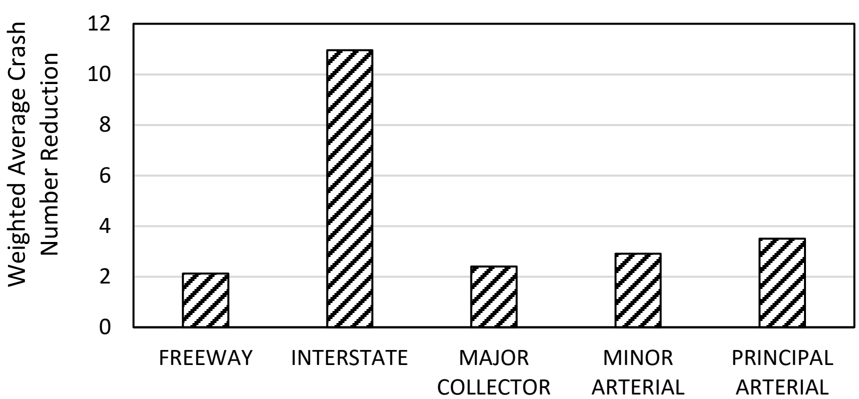


Figure 3-9: Reduction in crash numbers for different functional classes of pavements

Figure 3-10 shows a nine reduction in the weighted average crash number for major pavements after the application of HFST compared to the weighted average crash number observed before the application of HFST. For minor pavements, the weighted average reduction in crash number was observed to be 2.64. One of the factors that may have affected the higher reduction of crashes on major pavements may be speed limit and Annual Average Daily Traffic (AADT). The average speed limit on major pavements was 61.9 mph, while the average speed on minor pavements was 53.2 mph. The average AADT for major and minor pavements are 15,488 and 1,621, respectively. Therefore, the application of HFST and subsequent increase in friction number (FN) reduces the high-speed roadway departure crashes more under higher traffic conditions of major pavements compared to the minor pavements.

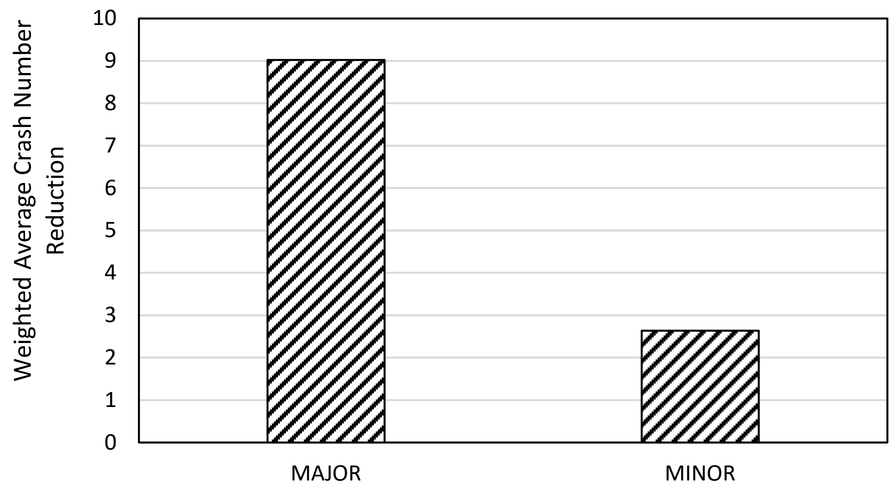


Figure 3-10: Reduction in crash numbers for major minor pavements

Figure 3-11 shows the reduction in crash numbers by area designation, rural or urban. It shows that in urban and urbanized areas, the application of HFST reduces the weighted average crash number by 12.7 and 10.3, respectively. On the other hand, a weighted average crash number reduction of 6.94 was observed in the rural pavements after the application of HFST. Thus, it can be said that the application of HFST is more effective in the urban and urbanized areas compared to the rural areas.

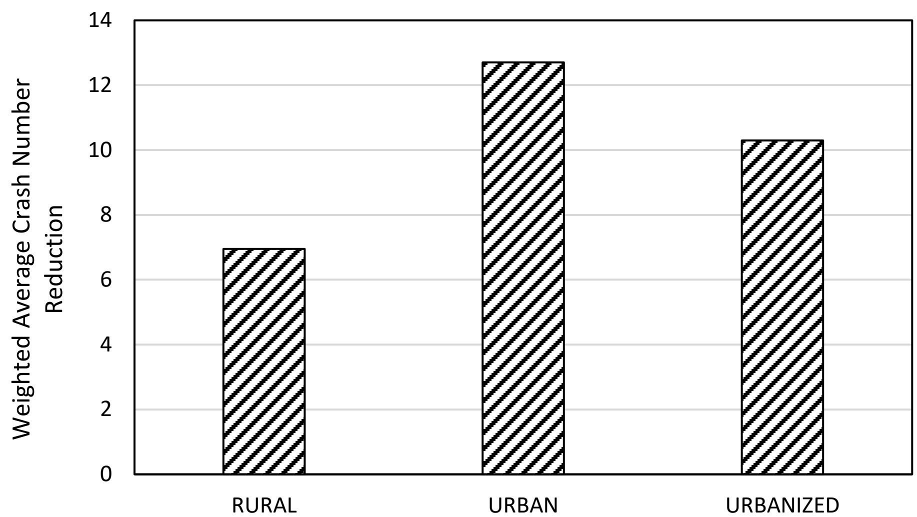


Figure 3-11: Reduction in crash numbers by designation of area

Figure 3-12 shows the reduction in the weighted average crash number based on the pavement surface type. As mentioned previously, the weighted average crash reduction was determined based on the AADT and length of the HFST sections. Figure 3-12 shows that except for AC and ultrathin bonded Type C (UTC), the application of HFST reduced crash numbers in all types of pavement surfaces. The highest reduction in crash numbers, around 12, is observed after the application of HFST was observed for Portland cement non-reinforced (PCN) and stone mastic AC mix (SM) type surfaces. After the application of HFST, the average crash number increases by 1 and 6 for AC and UTC types of pavement surfaces, respectively. The full forms of the surface abbreviations are provided in the Appendix A. It is important to mention that if the as-built plans with mix type were not available or just didn't show an SP designation, then the default mix type would be AC. In addition, AC might have been designated in lieu of bituminous pavement (BP) and surface level (SL) mixes, unless those were shown correctly.

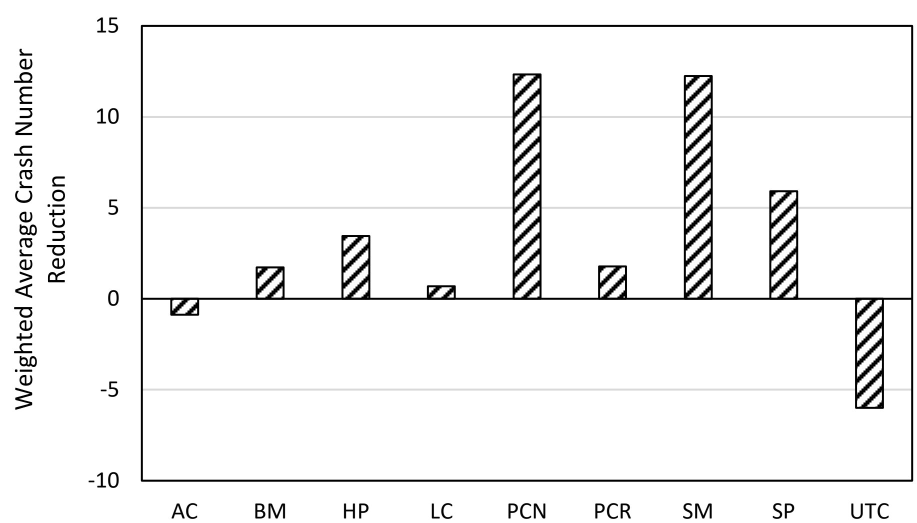


Figure 3-12: Reduction in crash numbers by surface type

Figure 3-13 shows a positive reduction in the weighted average crash number after the application of HFST, regardless of the age of the pavement surface. At least four crash reductions were observed after the HFST application. HFST has an average service life of about five years. Figure 3-13 shows that the most benefit, in terms of positive crash number reduction, is observed for pavements of less than five years of age. Even after five years, HFST can still provide a positive reduction in crash numbers, given rehabilitative work has not been conducted on the surface, or the HFST has not been removed.

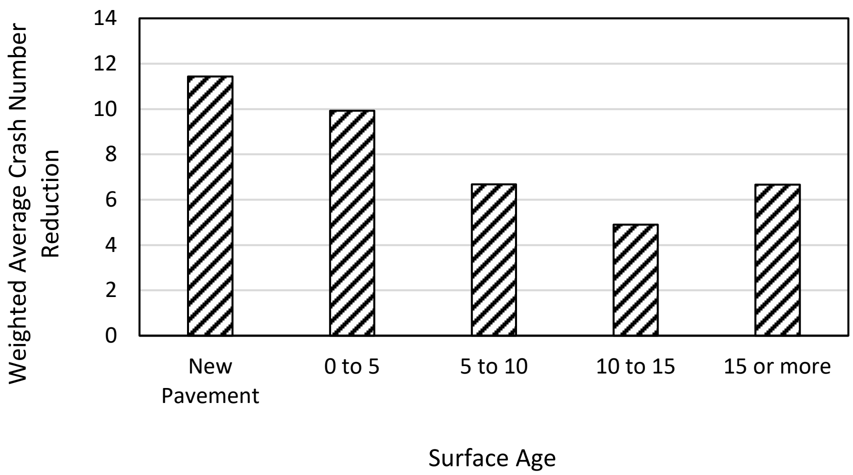
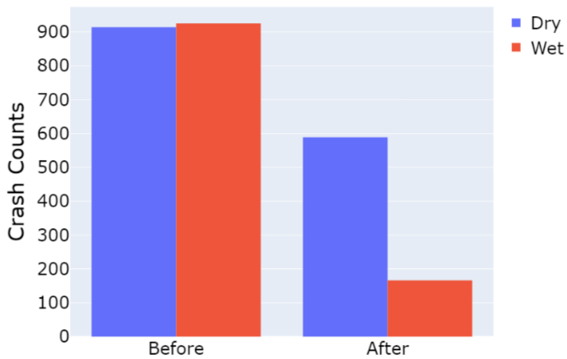
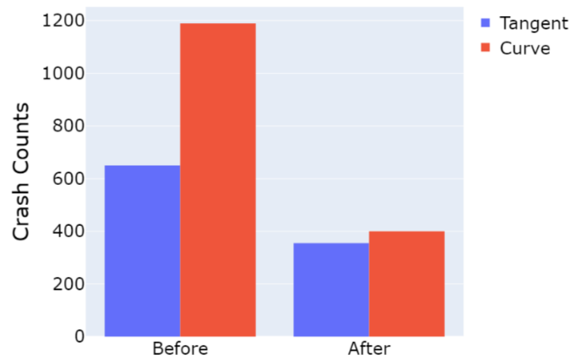


Figure 3-13: Reduction in crash number by surface age

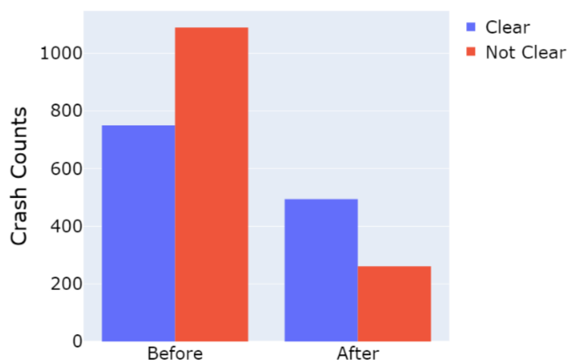
The second crash data file obtained from MoDOT had crash information divided by travel conditions, such as roadway conditions, alignment, weather conditions, etc. The file had crash data up to five years before and after the application of HFST. Figure 3-14 shows the crash counts before and after the application of HFST for various travel condition.



(a) Road condition



(b) Road alignment



(c) Weather condition

Figure 3-14: Crash count by travel condition

Figure 3-14 shows that HFST is an effective measure to reduce crashes in challenging driving conditions. Particularly, it is most effective in reducing crashes in wet conditions. Figure 3-14(a) shows that there were 925 wet crashes on the 146 segments prior to HFST. This was reduced to 166 in the post-application. However, only 48 of the 146 segments had reduced crashes post-construction, indicating that the large number of wet crashes was concentrated on a relatively small number of sections with large numbers of crashes. After wet conditions, HFST was found to be effective in reducing crashes when weather conditions are not clear or visibility is low, such as rain, snow, fog, etc.

3.5. SAFETY ANALYSIS

In the previous section, it was observed that the application of HFST is most beneficial in reducing the wet crash numbers. However, a safety analysis is needed to determine the actual Crash Reduction Factor (CRF) due to the application of HFST. CRF can be determined using the Crash Modification Factor (CMF). The main difference between CMF and CRF is that CMF is a multiplication factor used to determine the expected number of crashes, while CRF provides a percent reduction in crashes. The relation between CMF and CRF is shown in Eq. (2).

$$CMF = 1 - \frac{CRF}{100} \quad (2)$$

A safety performance function (SPF) is typically required to determine the CMF. In this study, crash data from 146 HFST sections were available. However, the development of SPF requires a larger database for both the model development and subsequent calibration. Moreover, the calibration requires detailed crash record reviews. Thus, this study developed a safety function based on the limited available data. It is important to note that the developed safety function is not an SPF in the traditional sense and only applies to the HFST sites considered in this study. The purpose of the developed safety functions was to determine the effectiveness of the application of HFST from CRF.

In this study, four safety functions were determined. Two for wet crashes and two for dry crashes.

3.5.1. Safety Functions for Wet Crashes

Two safety functions for wet crashes were developed based on the AADT. The first wet crash model was for AADT less than 10,000, and the second one was for AADT ≥10,000. The functional form of the crash number prediction model is shown in Eq. (3)

$$Predicted\ Crash = Length^{b_0} \times exp^{b_1 AADT} \times exp^{b_2 speed} \times exp^{b_3 BeforeAfter} \times exp^{a} \quad (3)$$

The Negative Binomial (NB) regression was used to fit the data in the above functional form. Table 3-2 lists the variables and their respective coefficients for the wet crash prediction model with AADT < 10000. All the variables in the model are continuous variables, except the variable 'BeforeAfter', which is a categorical variable that indicates the existence of the HFST, i.e., 0 indicates crashes observed before HFST while 1 indicates crashes observed after HFST.

Table 3-2: Coefficients of variables for wet crash model with AADT < 10000

Variable	Coefficient	Significant (p-value)
Intercept	3.7	No (0.835 > 0.05)
Length	0.397	No (0.112 < 0.05)
AADT	0.048	Yes (0.038 < 0.05)
Speed	-0.03	Yes (0.014 < 0.05)
BeforeAfter	-1.423	No (0.228 > 0.05)

Table 3-3 lists the variables and their respective coefficients for the wet crash prediction model with AADT ≥ 10000. All the variables in Eq. (3) are continuous variables except the variable 'BeforeAfter', which is a binary variable.

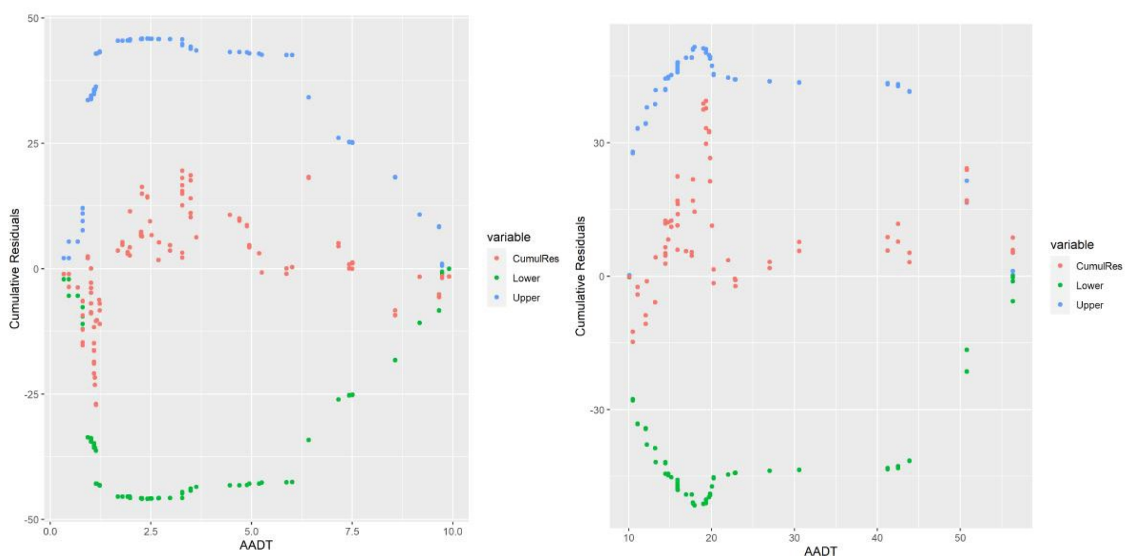
Table 3-3: Coefficients of variables for wet crash model with AADT ≥ 10000

Variable	Coefficient	Significant (p-value)
Intercept	4.22	No (0.778 > 0.05)
Length	0.672	No (0.136< 0.05)
AADT	0.004	Yes (0.006 < 0.05)
Speed	-0.017	Yes (0.011 < 0.05)
BeforeAfter	-1.3	No (0.164 > 0.05)

For both models, AADT and speed were found to be significant predictors of wet crash counts. Although the length of the HFST sections and the before-after variable were not found to be a significant predictor for the dataset considered, their inclusion improved the test statistics. After developing the safety functions, the goodness-of-fit or the test statistics of the NB regression model can be visually observed through the cumulative residual (CURE) plot. A good CURE plot has the following characteristics:

- Fluctuations around the horizontal (x-axis) axis and terminates around the value of zero
- No presence of outliers
- Minimum drifting in an upward or downward direction
- Residuals should rarely exceed the 95% upper or lower confidence interval

Figure 3-15 shows the CURE plots for both the wet crash prediction models. It can be seen that, for the most part, the CURE plots follow the characteristics outlined above. The maximum drift observed is less than ± 40 for both models.



(a) AADT < 10000 (b) AADT ≥ 10000

Figure 3-15: Cumulative residual plots for wet crash models

Once the goodness-of-fit of the developed models was visually verified, they were used to determine the CRF, as shown in Eq. (4). Eq. (5) and (6) show the CRF for the wet crash condition after the application of HFST.

$$CRF = \frac{Crash^{After}}{Crash^{Before}} = exp^{b_3(After - Before)} = exp^{b_3 After} \quad (4)$$

For wet crash in AADT < 10000,

$$CRF = exp^{-1.423} \quad (5)$$

For wet crash in AADT ≥ 10000,

$$CRF = exp^{-1.3} \quad (6)$$

3.5.2. Safety Functions for Dry Crashes

Similar to the wet condition, crash prediction models were also developed for the dry condition with the same functional form shown in Eq. (3). Table 3-4 shows the coefficients and p-values of the variables for the dry crash prediction models with AADT < 10000.

Table 3-4: Coefficients of variables for dry crash model with AADT < 10000

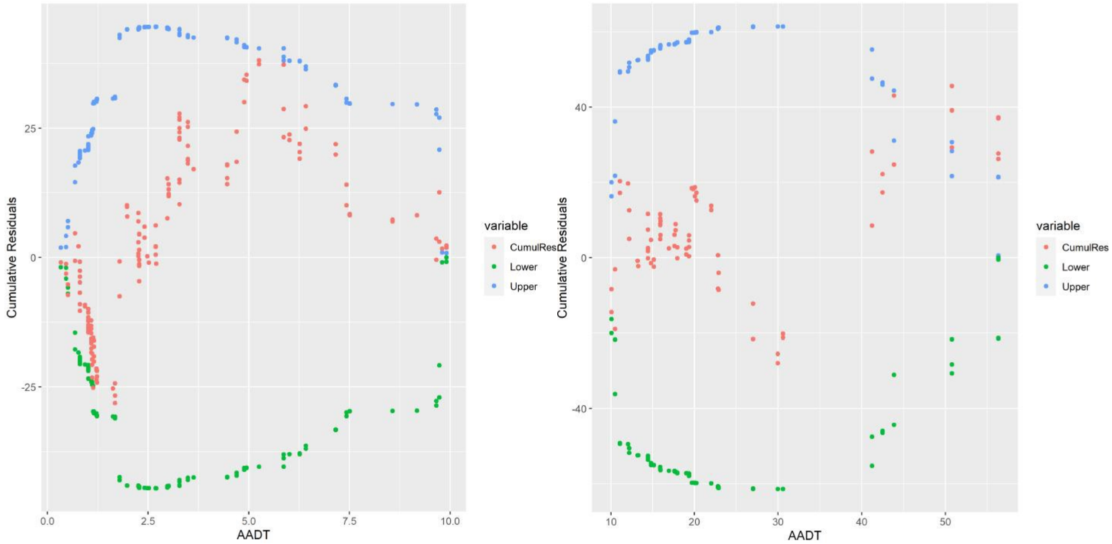
Variable	Coefficient	Significant (p-value)
Intercept	4.269	No (0.611 > 0.05)
Length	0.509	No (0.084 < 0.05)
AADT	0.085	Yes (0.026 < 0.05)
Speed	-0.04	Yes (0.01 < 0.05)
BeforeAfter	-0.386	No (0.132 > 0.05)

Table 3-5 shows the coefficients and p-values of the variables for the dry crash prediction models with AADT ≥ 10000. Similar to earlier models, AADT and speed were found to be significant predictors.

Table 3-5: Coefficients of variables for dry crash model with AADT ≥ 10000

Variable	Coefficient	Significant (p-value)
Intercept	6.755	No (0.774 > 0.05)
Length	1.021	No (0.139 < 0.05)
AADT	0.025	Yes (0.006 < 0.05)
Speed	-0.065	Yes (0.011 < 0.05)
BeforeAfter	-0.348	No (0.154 > 0.05)

Figure 3-16 shows the CURE plots for the dry crash prediction models. Although few points at the end exceeded the upper limits in the CURE plot shown in Figure 3-16(b) for AADT ≥ 10000, the overall trend followed the characteristics mentioned previously, and the residuals fluctuated between ±40.



(a) AADT < 10000 (b) AADT ≥ 10000

Figure 3-16: Cumulative residual plots for dry crash models

The CRFs for dry conditions are shown in Eq. (7) and (8). For dry crash in AADT < 10000,

$$CRF = exp^{-0.386} \tag{7}$$

For dry crash in AADT ≥ 10000,

$$CRF = exp^{-0.348} \tag{8}$$

Table 3-6 shows the CRF for different driving conditions. It shows that the average reduction in wet crashes due to the application of HFST is 74.3%. In dry conditions, the average crash reduction is observed to be 30.7%. Therefore, HFST is found to be most effective in reducing crashes in wet conditions based on the CRF from crash data analysis.

Table 3-6: CRF for Different driving conditions

Condition	CRF (%)		
	AADT < 10000	AADT ≥ 10000	Average
Wet	75.9	72.7	74.3
Dry	32	29.4	30.7

It is important to mention that the safety analysis presented in this chapter does not distinguish between different roadway types. It is included to provide a general idea regarding the effectiveness of HFST on various roadway conditions before and after the application of the treatment, as well as the test statistics that need to be checked during the development of SPF. A more detailed and appropriate procedure for safety analysis will be discussed in the next chapter.

3.6. STATISTICAL TESTS

CRF analysis showed that the application of HFST can reduce crashes in wet and dry conditions. Based on the available data, a statistical analysis was conducted in this section to determine if the reduction in crash numbers before and after the application of HFST in dry or wet conditions is statistically significant or not. Figure 3-17 shows the distribution of crashes observed per HFST sections in dry or wet conditions before and after application of HFST.

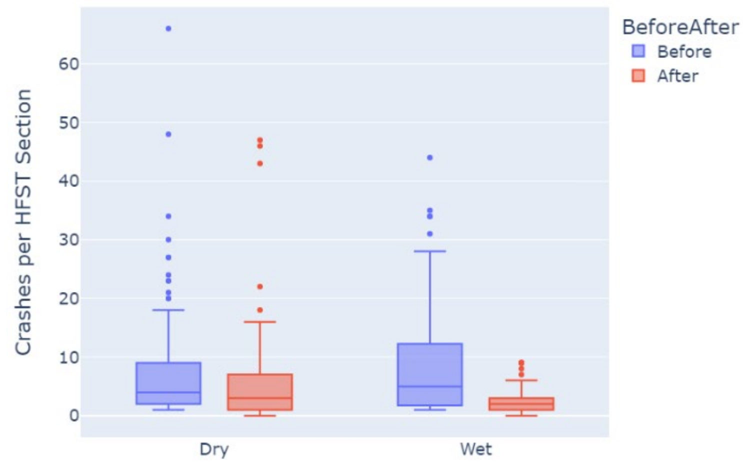


Figure 3-17: Observed crashes before and after application of HFST in dry or wet conditions

A statistical t-test was conducted to determine if the crashes observed before and after HFST were significantly different or not. The hypotheses of the t-test are as follows:

- Null hypothesis: the means between the groups are equal, i.e., there is no significant difference
- Alternate hypothesis: the means between the groups are different, i.e., there is a significant difference

For the dry condition, the p-value was found to be 0.07. As this value is greater than the threshold value of 0.05, we cannot reject the null hypothesis. Thus, it can be said with 95% confidence that the crashes observed in dry conditions before and after the application of HFST are not statistically different. However, for wet conditions, the p-value was observed to be $1.23\times10^{\wedge -6} < 0.05$. It implies that there is a statistically significant difference in the crashes observed in wet conditions before and after application of HFST.

4. GUIDANCE ON HFST

4.1. INTRODUCTION

The initial objective of this study was to provide guidance on how the observed distresses prior to the application of an HFST could affect the success of that treatment and the overall safety benefit of such projects. However, due to the challenges associated with distress data discussed previously in this report, we are suggesting the development of guidelines based on the combination of data, engineering judgment, and findings of the performed literature review.

The following flow chart shows an overall method that might be used to document the selection of HFST projects from beginning to end. Figure 4-1 shows such a general decision flow.

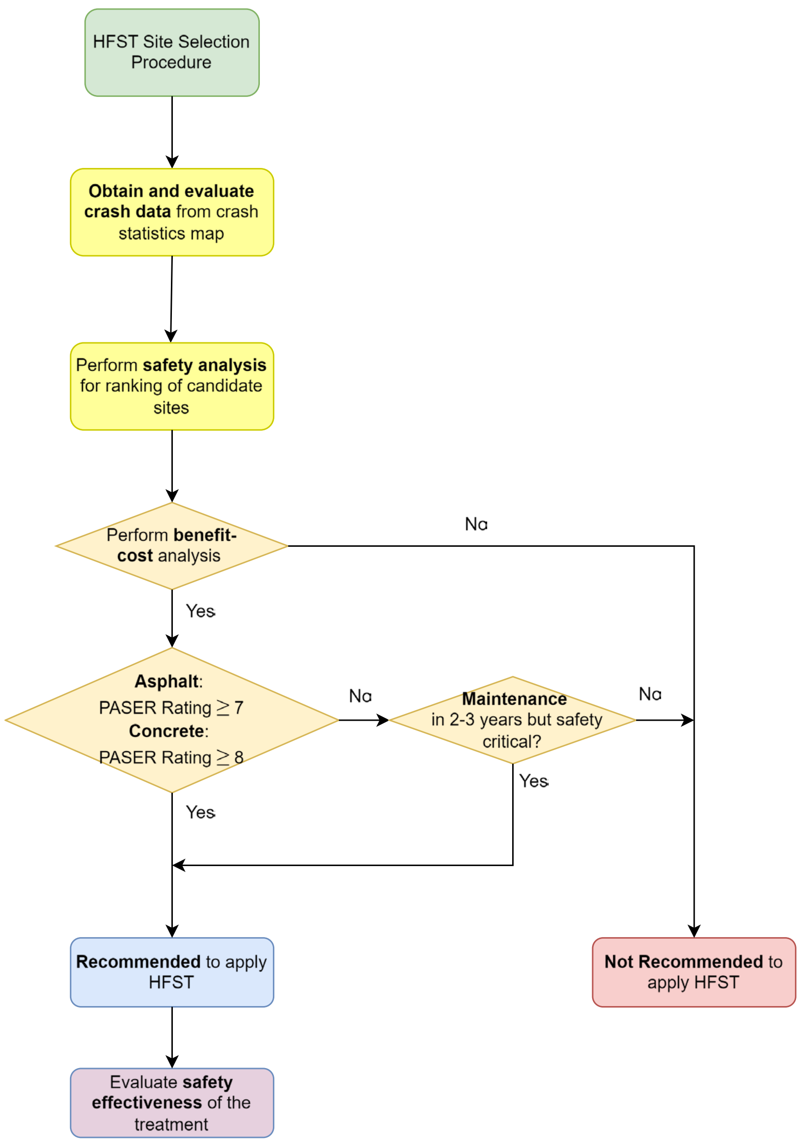


Figure 4-1: Flowchart for guidelines on HFST application

Each step in the Chart will be discussed, and examples included where possible to explain the process.

4.2. OBTAIN AND EVALUATE CRASH DATA

There are several possible methods to select locations for the application of HFSTs. The first and most obvious is simply the observation of recurring crashes at specific locations. This data may come from MoDOT staff, law enforcement, or the public. While this is reactive, it is certainly a valid method to begin an investigation.

A second method is the use of data on a more expansive system-wide basis. MoDOT is well equipped to develop a more proactive plan using the system, condition, and crash data available in its TMS database.

One way this can be done is through the Crash Statistics Map application available through the Datazone. A screenshot of the crash statistics map application is shown in Figure 4-2. This application allows the user to view corridors, counties, specific routes, or larger areas with regard to historical crash information.

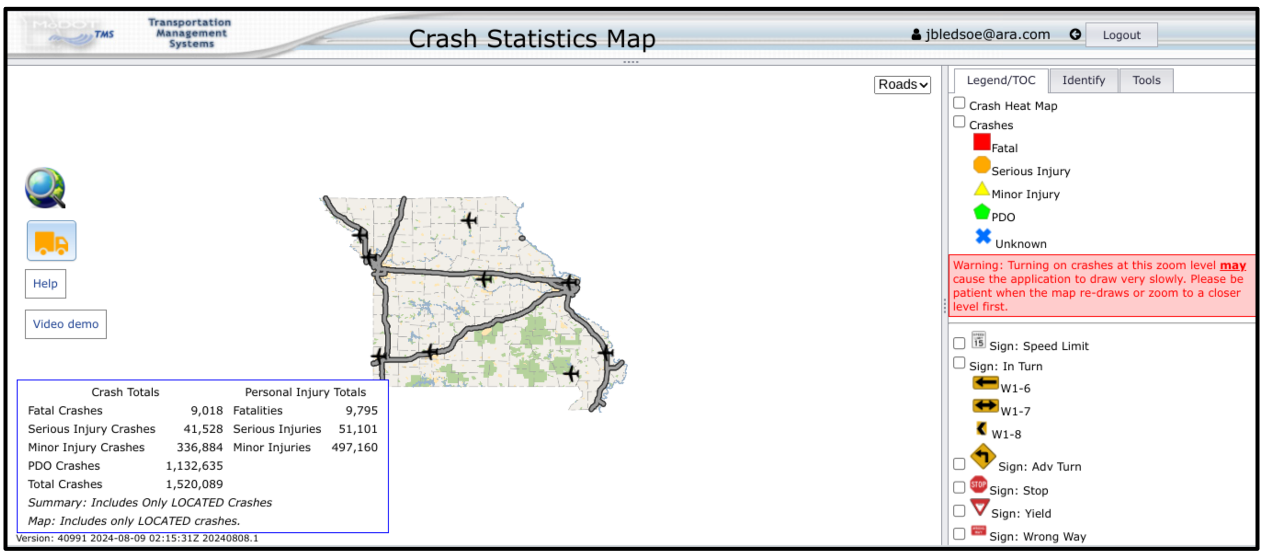


Figure 4-2: Crash statistics map application (source: MoDOT)

Two examples of utilizing crash maps to obtain crash data are provided based on actual locations used in this study. In example 1, Figure 4-3 shows crash data displayed on a map using this application.

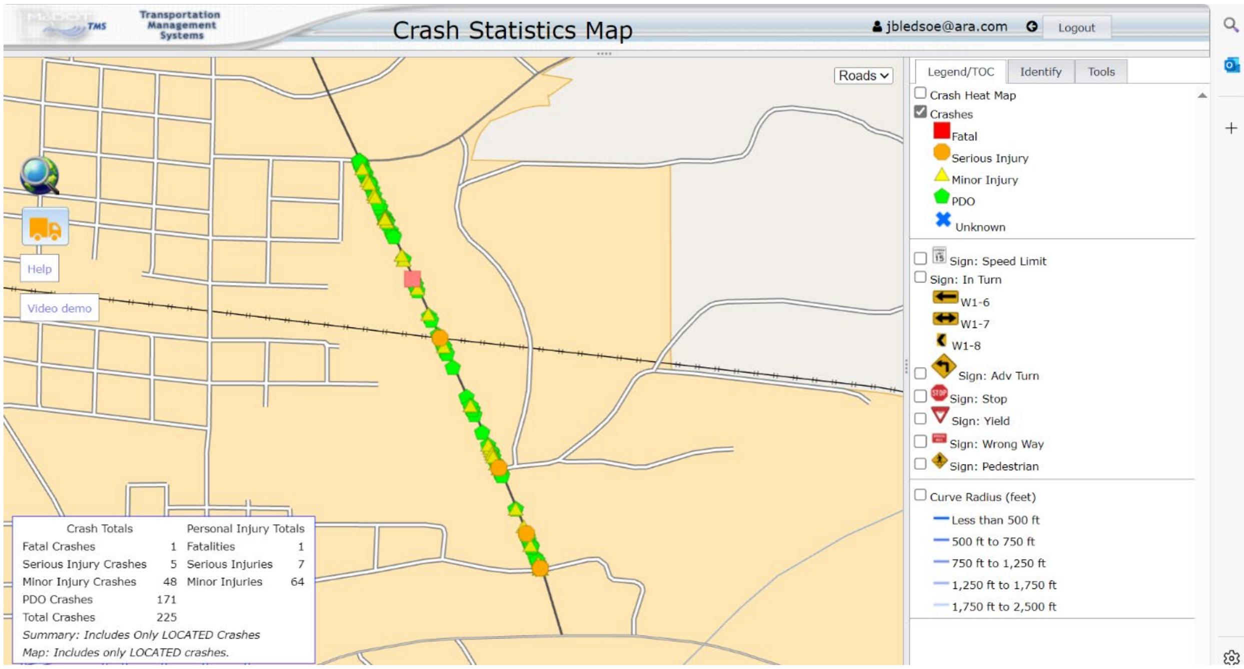


Figure 4-3: Map of total crash data (source: MoDOT)

The data shown includes all crashes in the historical database for this area. At first glance, this area may seem to show a location with an above-average crash history. However, if the data is filtered to display only the previous 3-years of crash history, the result is significantly different, as shown in Figure 4-4.

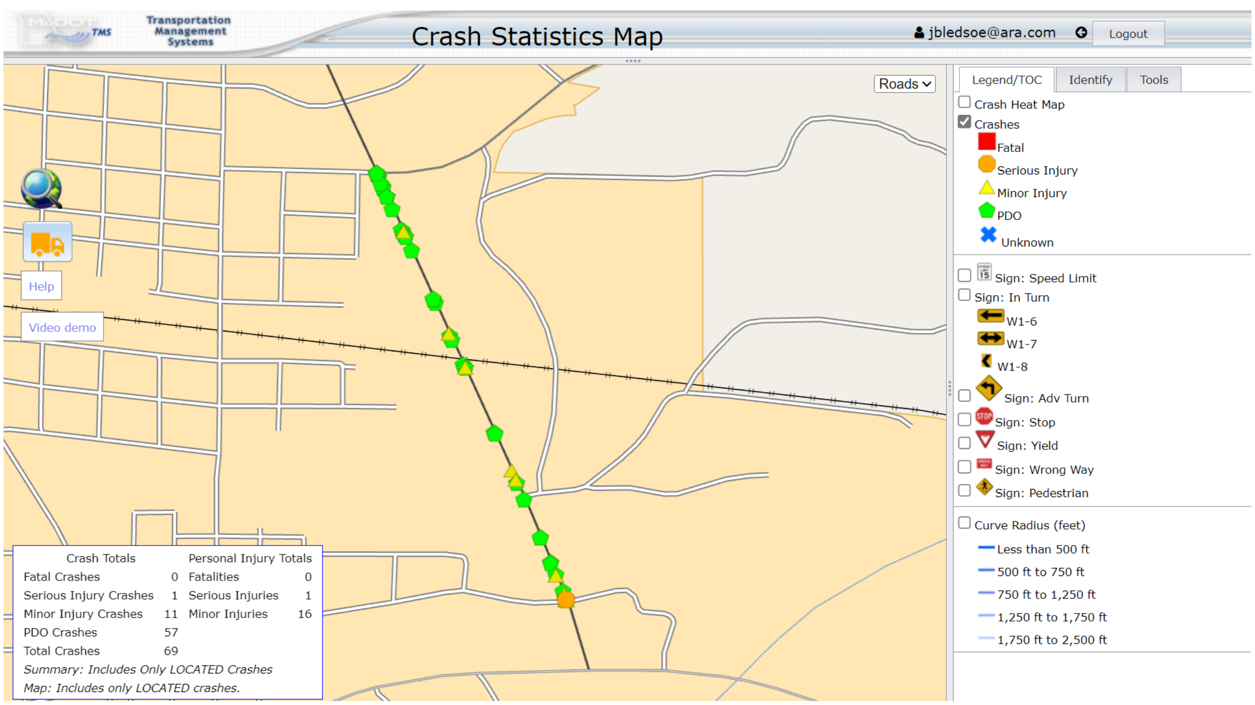


Figure 4-4: Crash history for a period of three years (source: MoDOT)

Looking at only 3-years of data, the number of crashes that occurred on the section being investigated drops from 225 to 69. Figure 4-5 shows that further filtering of data to include only wet crashes reduces the number in the subject area to only 8.

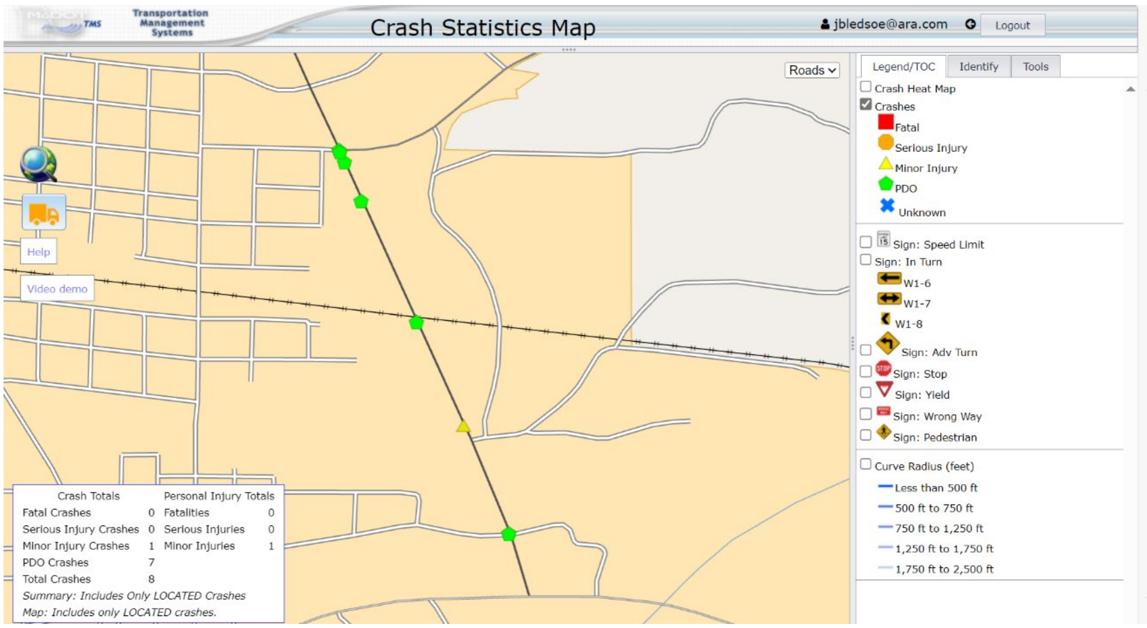


Figure 4-5: Wet crash history for three-year period (source: MoDOT)

Given the expected drop in crashes that can be attributed to HFST developed in this report, one could expect the dry crashes to drop from 61 to 43 and the wet crashes from 8 to 2. In reality, as can be seen in Figure 4-6, the total crashes were reduced to 43 and the wet crashes to 7 (Figure 4-7).

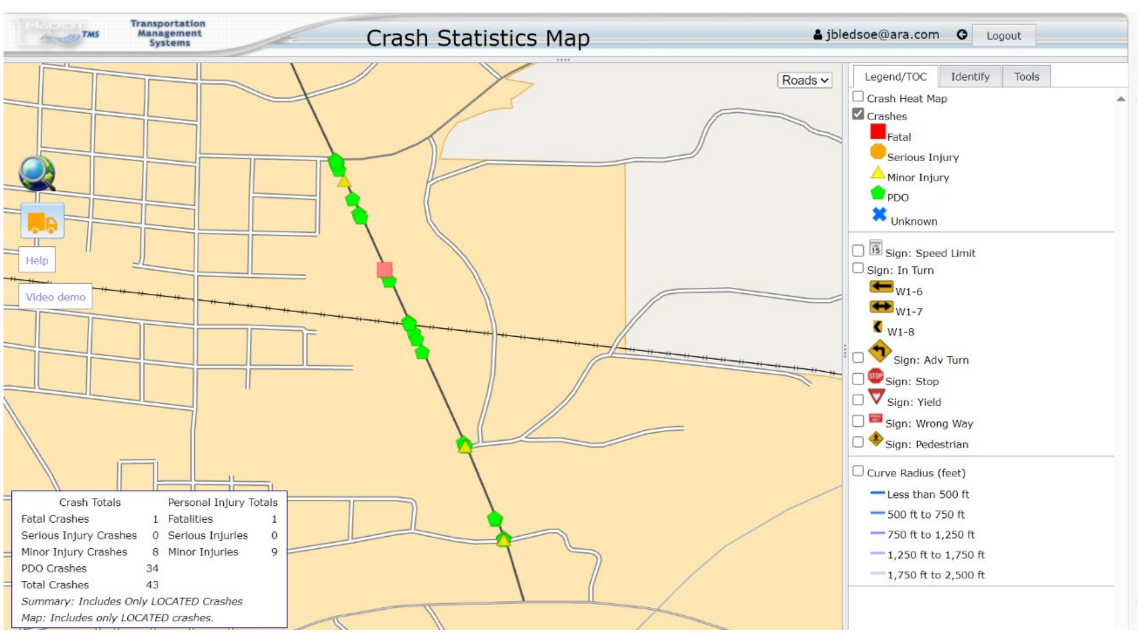


Figure 4-6: Total crash history 3-years following application (source: MoDOT)

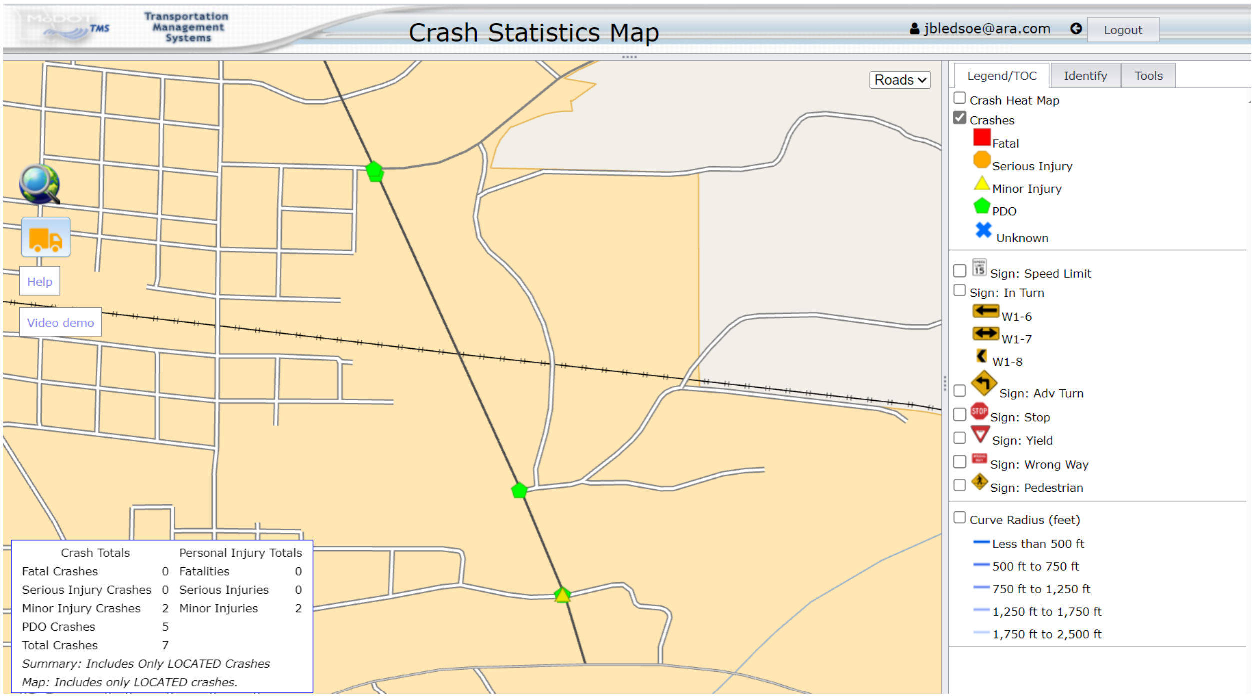


Figure 4-7: Total wet crashes 3-years after application (source: MoDOT)

Example 2 of crash map utilization is also taken from the dataset used for this study. In this case, the data shows a significant number of crashes occurring at a curve section, as shown in Figure 4-8.

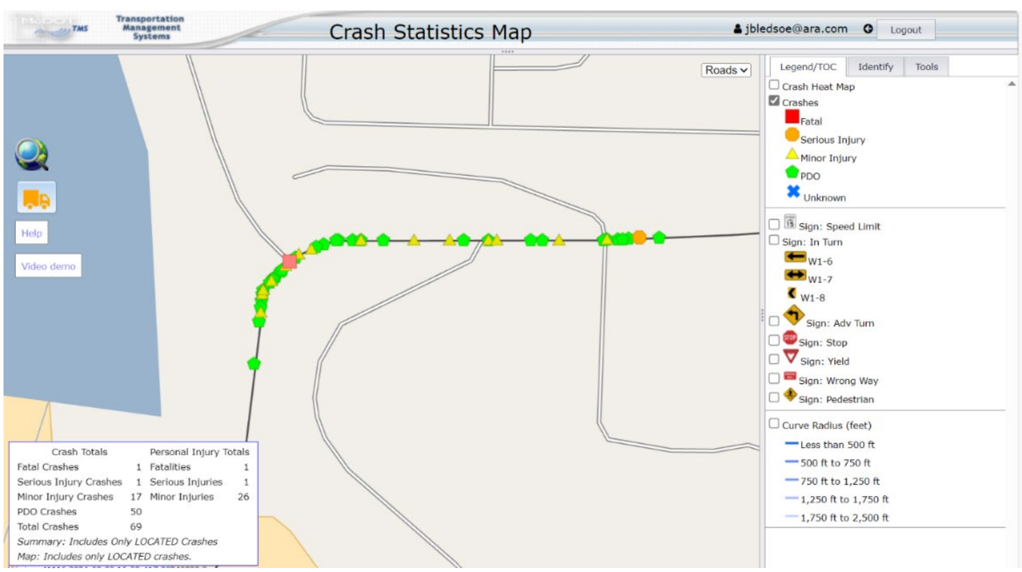


Figure 4-8: Total crash history (source: MoDOT)

In the initial review, 69 crashes are shown within the limits of the proposed project. When a filter is applied to limit the crashes to 3-years prior to the application, the number drops to 35, as shown in Figure 4-9.

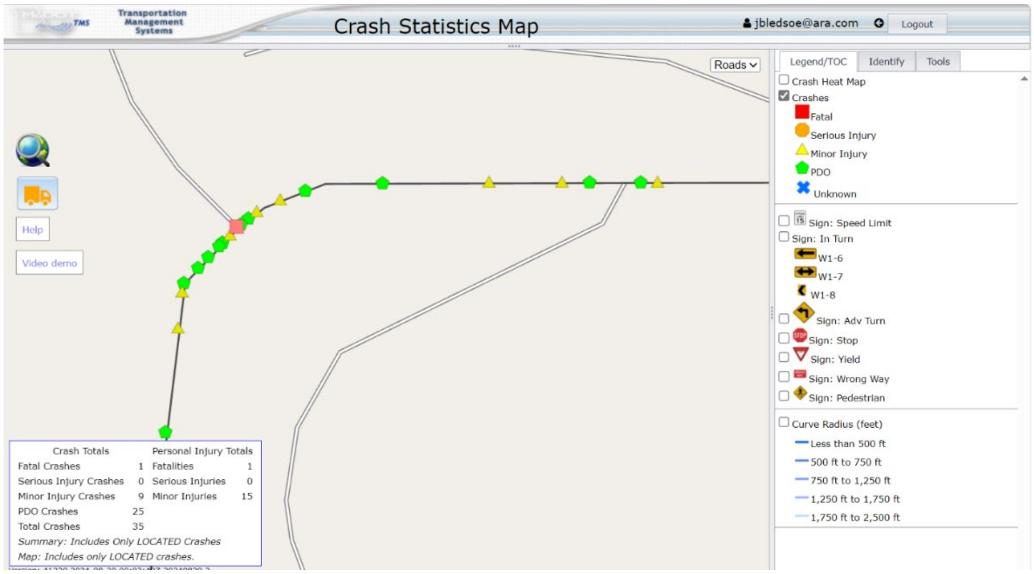


Figure 4-9: Total crash history for three-year period (source: MoDOT)

A look at the distribution of wet and dry crashes in Figure 4-10 shows 23 wet crashes and 12 under dry conditions.

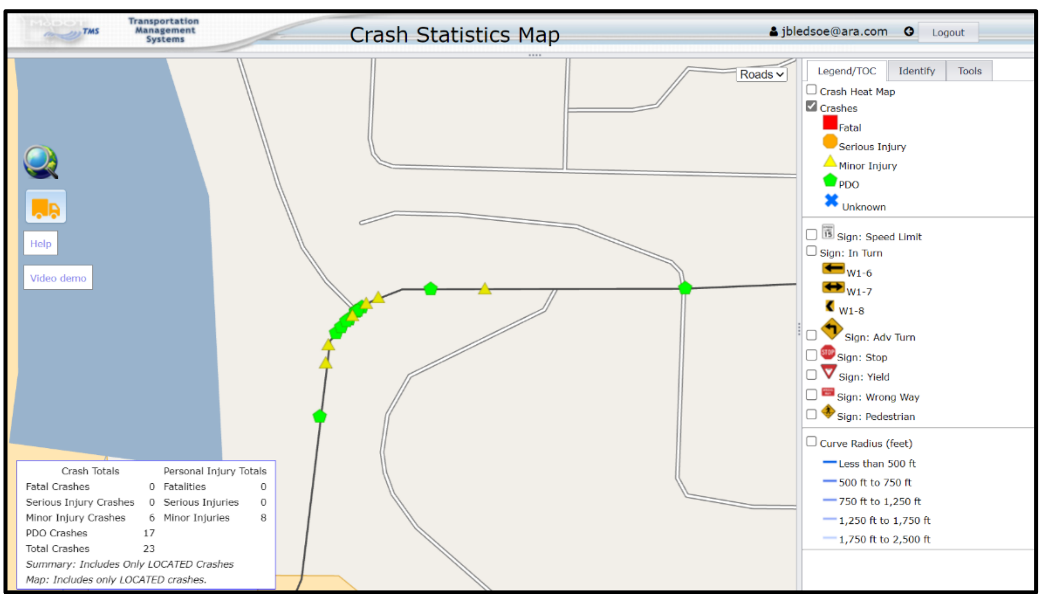


Figure 4-10: Wet crash history for three-year period (source: MoDOT)

If we look at the average reduction that might occur following the application, using the network-derived averages obtained in the previous chapter, we could expect the total number of wet crashes to drop from 23 to approximately 7 (70 percent reduction) in 3-years following the application. Likewise, the dry crashes should be reduced by about 30 percent to approximately 8.

In this case, there were only two wet crashes in the following 3-year period, and the total crashes for the 3-years following the application was reduced to 16 from 35. While these examples show the inaccuracy of using network-derived averages to predict future crashes, they can also be indicators of the magnitude of benefits that might be achieved. Although the evaluation of crash numbers from the crash map provides an indication of apparent benefits, an appropriate safety analysis is required to predict the network-level benefits.

4.3. SAFETY ANALYSIS

After the initial selection of several candidate sites based on the crash data, a safety analysis should be performed. The following steps can be used to determine the ranking of the candidate sites:

- Determination of the predicted average crash frequency using the SPF.
- Calculation of the annual correction factors for each considered year and crash severity.
- Calculation of the weightage factor using the overdispersion parameter and predictive crash number from the SPF.
- Using the Empirical Bayes (EB), adjust the expected average crash frequency for the first and the final year.
- Rank the sites based on the crash numbers.

After ranking the candidate sites based on the expected number of crashes, a number of sites should be selected to apply the HFST to observe their effectiveness. Further details regarding the above-mentioned procedures can be found in Highway Safety Manual (HSM) (AASHTO, 2009).

It is important to note that both safety effectiveness evaluation and ranking of sites according to expected crash numbers require SPF for the appropriate facility type, and the predicted average crash frequencies are applicable to network-level analysis. It is recommended that MoDOT develop SPFs for different facility types and the severity of crashes for a robust and reliable safety analysis. MoDOT may also elect to use the SPF developed in other jurisdictions, such as the ones developed in HSM, using national data. However, HSM recommends calibrating the SPFs using data from the jurisdiction where the predictive models will be used (AASHTO, 2009). According to Appendix A of Part C of HSM, the calibration process of SPF is as follows:

- Identification of the appropriate facility types for which the predictive model will be calibrated.
- Selection of an adequate number of sites for the calibration of the predictive models of each facility type.
- Obtain the crash data for the applicable calibration period for each facility type.
- Use the predictive models to predict the total crash frequency for each site for the calibration period with appropriate crash modification factors.
- Using the summation of total observed crashes and total predicted crashes for all the sites for a particular facility type to determine their ratio or calibration factor.

A step-by-step example calculation involved in the calibration of SPF, ranking of sites, and effectiveness of the application of HFST for two-way, two-lane, undivided rural roadway segments with horizontal curves are provided below.

4.3.1. Calibration of SPF

According to HSM, Eq. (9) shows the SPF for a two-lane two-way undivided rural road.

$$N_{SPF\ RS} = AADT \times L \times 365 \times 10^{-6} \times e^{-0.312} \times AMF_i \tag{9}$$

Table 4-1 shows the AADT and length of the HFST sections that are located on the two-lane two-way rural undivided roadways with a horizontal curve. Accidents Modification Factors (AMFs) are based on the roadway characteristics of each site. In this study, lane width, shoulder width, curve length, and radius were available for each HFST sites. AMFs for these roadway characteristics were determined according to the Part C predictive method of HSM (AASHTO, 2009). Using Eq. (9), crashes were also predicted for each HFST section for the base condition. The total predicted crashes for this type of facility were calculated to be 130.43. The total amount of crashes that were observed for the considered facility in the available time period was 634. Now, according to HSM, the calibration factor for the SPF can be determined using Eq. (10).

$$C = \frac{\sum_{all\ sites} Observed\ Crashes}{\sum_{all\ sites} Predicted\ Crashes} \tag{10}$$

According to HSM, 30 to 50 sites experiencing at least 100 crashes per year are required for the calibration of SPFs. Using Eq. (10), the calibration factors for the two-lane two-way undivided rural road were determined to be 4.86.

Table 4-1: Crashes in two lane two way undivided rural roads with horizontal curve

HFST Section	AADT	Length (mile)	Observed Crashes	Time (years)	Lane Width (ft)	Shoulder Width (ft)	Curve Length (ft)	Curve Radius (ft)	Lane Width AMF	Shoulder Width AMF	Horizontal Curve AMF	SPF Predicted Crash
18	1976	0.4	28.0	9	11.0	6	1207.1	1158.4	1.05	1.00	1.16	2.08
19	1925	0.1	2.0	2	11.0	6	574.3	741.4	1.05	1.00	1.57	0.10
31	1091	0.3	7.0	5	11.0	2	558.2	887.5	1.03	1.17	1.48	0.75
34	1091	0.1	3.0	3	11.0	2	579.3	806.5	1.03	1.17	1.51	0.14
35	1091	0.1	4.0	4	11.0	2	742.7	962.3	1.03	1.17	1.33	0.23
38	1091	0.1	2.0	2	11.0	2	589.3	808.9	1.03	1.17	1.50	0.11
50	8569	1.2	33.0	10	12.0	8	1500.3	3029.3	1.00	0.87	1.03	23.68
56	2981	1.9	41.0	9	12.0	10	2908.1	3522.6	1.00	0.87	1.01	12.05
64	1792	0.2	47.0	10	10.5	2	345.8	229.9	1.15	1.27	4.32	6.90
69	4886	0.5	32.0	10	10.0	3	1174.4	1201.3	1.30	1.23	1.16	12.31
83	688	0.1	19.0	7	11.0	3	391.6	253.8	1.02	1.08	3.64	0.46
88	4461	0.1	17.0	8	10.0	2	455.4	304.4	1.30	1.30	2.88	6.82
89	4461	0.2	4.0	4	10.0	2	714.1	575.8	1.30	1.30	1.61	2.24
103	1137	0.3	3.0	3	11.0	2	1233.9	2935.2	1.03	1.18	1.04	0.31
104	1137	0.1	34.0	7	11.0	2	793.3	2693.9	1.03	1.18	1.08	0.27
111	1684	0.3	27.0	6	10.0	3	642.5	1405.7	1.24	1.19	1.24	1.59
151	807	0.2	2.0	2	11.0	2	1259.7	703.1	1.02	1.13	1.28	0.15
152	807	0.2	5.0	5	11.0	2	527.5	315.4	1.02	1.13	2.56	0.48
160	807	0.6	10.0	5	11.0	2	588.9	1213.3	1.02	1.13	1.31	0.96
171	1014	0.2	4.0	3	11.0	2	528.1	877.3	1.03	1.16	1.51	0.25
174	1014	0.2	11.0	6	11.0	2	483.1	724.5	1.03	1.16	1.70	0.56
175	1014	0.1	5.0	5	11.0	2	454.3	373.6	1.03	1.16	2.52	0.36
205	2258	0.3	10.0	7	11.0	4	1899.9	2789.7	1.05	1.15	1.03	1.74
208	2258	0.2	9.0	6	11.0	4	1362.8	1157.5	1.05	1.15	1.14	1.17
209	2690	0.4	18.0	7	11.0	4	2066.4	1972.5	1.05	1.15	1.05	2.35
212	3283	0.2	7.0	6	12.0	5	680.9	756.7	1.00	1.08	1.47	1.34

Table 4-1: continued

HFST Section	AADT	Length (mile)	Observed Crashes	Time (years)	Lane Width (ft)	Shoulder Width (ft)	Curve Length (ft)	Curve Radius (ft)	Lane Width AMF	Shoulder Width AMF	Horizontal Curve AMF	SPF Predicted Crash
213	3283	0.7	41.0	9	12.0	5	612.8	840.2	1.00	1.08	1.46	8.19
214	3283	1.3	47.0	10	12.0	5	592.3	670.9	1.00	1.08	1.62	19.94
215	3283	0.1	7.0	6	12.0	5	571.7	641.3	1.00	1.08	1.67	1.30
217	3283	0.1	5.0	5	12.0	5	539.1	855.1	1.00	1.08	1.52	0.58
238	3629	0.3	3.0	3	12.0	6	902.0	2077.7	1.00	1.00	1.10	1.03
243	2705	0.2	9.0	6	11.0	3	680.2	595.1	1.05	1.23	1.61	1.63
252	935	0.3	33.0	8	11.0	2	514.2	805.3	1.02	1.15	1.58	1.29
258	2412	0.2	11.0	5	11.0	3	995.0	581.3	1.05	1.23	1.43	1.39
260	1216	0.1	5.0	3	11.0	3	483.2	501.9	1.03	1.14	2.04	0.33
265	2282	0.4	14.0	7	11.5	6	760.1	1602.2	1.03	1.00	1.17	1.99
267	2282	0.2	23.0	6	11.5	6	633.5	797.9	1.03	1.00	1.48	1.03
268	2282	0.1	16.0	6	11.5	6	451.1	427.8	1.03	1.00	2.33	1.00
278	3492	0.3	33.0	11	11.0	2	576.9	370.8	1.05	1.30	2.21	10.45
279	3492	0.1	3.0	3	11.0	2	631.3	608.4	1.05	1.30	1.65	0.85
Summation	-	-	634.0	-	-	-	-	-	-	-	-	130.43
Calibration Factor	4.86											

It is important to mention that the above example of SPF calibration considered only the base condition, which is seldom the case. As stated earlier, the yearly crash predictions from the HSM-provided SPF for a certain facility type need to be multiplied by the appropriate calibration factors (C_i), as shown in Eq. (11).

$$N_{Predicted\ -rs} = N_{SPF-rs} \times C_i \times AMF_i \tag{11}$$

here, $N_{Predicted-rs}$ is prediction of crashes for the roadway segment after multiplying with appropriate C_i, and accident modification factors (AMF), N_{SPF-rs} is the SPF for base condition.

For crash prediction with a calibrated SPF, the following information regarding the roadway geometry and design is required to determine the appropriate AMFs:

- Lane width
- Shoulder width and type
- Roadside hazard rating
- Characteristics of horizontal curves: length, radius, presence of spiral transition
- Horizontal curves: superelevation
- Grades
- Driveway density
- Centerline rumble strips
- Passing lanes
- Two-way left turn lanes
- Lighting
- Automated speed enforcement

For each of the above roadway geometry and design, there is an accident modification factor. The base SPF, N_{SPF-rs}, shown in Eq. (9) and (11) is applicable for the following base conditions:

- Shoulder type = paved
- Roadside hazard rating = 3
- Driveway density = 5 driveways per mile
- Center line rumble strips = none
- Passing lanes = none
- Two-way left turn lanes = none
- Lighting = none
- Automated speed enforcement = none
- Grade level = none

For the base conditions, AMFs are 1. As stated earlier, modifications to any of the above base conditions will require an appropriate AMF to be multiplied with the calibrated SPF for accurate crash prediction. Along with the crash data, the above-mentioned information is also required for the calibration of SPF for other facility types. In this study, it is assumed that the spiral transition curve is present at both ends of the horizontal curve, and superelevation variance is less than 0.01, i.e., the difference between the superelevation rate according to AASHTO Green Book and the actual provided superelevation is less than 0.01.

4.3.2. Ranking of Sites

Once an SPF is developed or calibrated for a particular facility type, it can be used to rank candidate sites based on available safety data. As an example, the earlier calibrated SPF for the two-lane two-way undivided rural road is selected. Seven candidate sites were selected randomly from the available 40 HFST locations of the two-lane two-way undivided rural road with a horizontal curve. The ranking was conducted for a three-year period. For the candidate sites, the following data should be known: AADT, length of the section, and observed crashes for the period considered. Table 4-2 shows the candidate sites with the required information about the sites.

Step 1: Calculate the predicted average crash frequency and correction factor

Crash predictions ($N_{predicted}$) for each year are obtained using the AADT and length of candidate sites and Eq. (11). The correction factor (C_n) is obtained using Eq. (12)

$$C_n = \frac{N_{predicted,n}}{N_{predicted,1}} \tag{12}$$

where, $N_{predicted,n}$ predicted number of crashes in year *n*, $N_{predicted,1}$ is the predicted number of crashes in the first year.

For instance, for candidate site 1 in year 1, the predicted crash is shown in Eq. (13)

$$N_{SPF\ RS} = 1976 \times 0.36 \times 365 \times 10^{-6} \times e^{-0.312} \times 1.05 \times 1 \times 1.16 \times 4.86 = 1.13 \tag{13}$$

As AADT and length did not change for candidate site 1 for the subsequent years, the crash prediction remained the same. The calibration factor for year 3 for site 1 is determined as shown in Eq. (14)

$$C_3 = \frac{N_{predicted,3}}{N_{predicted,1}} = \frac{1.13}{1.13} = 1 \tag{14}$$

Eq. (15) is used to determine the overdispersion parameter for the SPF of two-way two lane undivided rural road. Candidate section 1 has a total length of 0.36 miles. Thus, according to Eq. (16), the value of overdispersion (*k*) was determined to be 0.66.

$$k = \frac{0.236}{L} \tag{15}$$

$$k_1 = \frac{0.236}{0.36} = 0.66 \tag{16}$$

Table 4-2: Crash predictions for the candidate sites for a three-year period

Candidate Section	Year	AADT	Length (*L*)	AMF_{LW}	AMF_{SW}	AMF_{Curve}	Predicted Crash from SPF ($N_{Predicted}$)	Correction Factor (*C*)	Overdispersion (*k*)
1	1	1976	0.36	1.05	1	1.16	1.13	1	0.66
	2	1976	0.36	1.05	1	1.16	1.13	1	0.66
	3	1976	0.36	1.05	1	1.16	1.13	1	0.66
2	1	1925	0.06	1.05	1.00	1.57	0.25	1	3.93
	2	1925	0.06	1.05	1.00	1.57	0.25	1	3.93
	3	1925	0.06	1.05	1.00	1.57	0.25	1	3.93
3	1	1792	0.228	1.15	1.27	4.32	3.36	1	1.04
	2	1792	0.228	1.15	1.27	4.32	3.36	1	1.04
	3	1792	0.228	1.15	1.27	4.32	3.36	1	1.04
4	1	4886	0.511	1.30	1.23	1.16	5.98	1	0.46
	2	4886	0.511	1.30	1.23	1.16	5.98	1	0.46
	3	4886	0.511	1.30	1.23	1.16	5.98	1	0.46
5	1	688	0.09	1.02	1.08	3.64	0.32	1	2.62
	2	688	0.09	1.02	1.08	3.64	0.32	1	2.62
	3	688	0.09	1.02	1.08	3.64	0.32	1	2.62
6	1	4461	0.147	1.30	1.30	2.88	4.15	1	1.61
	2	4461	0.147	1.30	1.30	2.88	4.15	1	1.61
	3	4461	0.147	1.30	1.30	2.88	4.15	1	1.61
7	1	1137	0.096	1.03	1.18	1.08	0.18	1	2.46
	2	1137	0.096	1.03	1.18	1.08	0.18	1	2.46
	3	1137	0.096	1.03	1.18	1.08	0.18	1	2.46

Step 2: Calculate weighting factor for Empirical Bayes (EB)

Once the overdispersion parameter (k) and predicted average crash frequency ($N_{predicted}$) are determined, Eq. (17) can be used to determine the weight for the Empirical Bayes (EB) to account for the return to the mean bias.

$$w = \frac{1}{1 + k \times \sum_{n-1}^{N} N_{predicted\ ,n}} \tag{17}$$

With an overdispersion parameter of 0.66 and a total predicted average crash frequency of 4.21 in the three-year period for candidate section 1, the EB weight (w_1) was determined to be 0.27, as shown in Eq. (18).

$$w_1 = \frac{1}{1 + k \times \sum_{n-1}^{N} N_{predicted\ ,n}} = \frac{1}{1 + 0.66 \times 3.38} = 0.31 \tag{18}$$

Step 3: Calculate EB adjusted average crash frequency and variance

Observed crash, predicted average crash frequency, EB weight, and the correction factor can be used to determine the EB-adjusted expected average crash frequency and variance as shown in Eq. (19) to (21).

$$N_{expected\ ,1} = w \times N_{predicted\ ,1} + (1 - w) \times \left(\frac{\sum_{n-1}^{N} N_{observed}}{\sum_{n-1}^{N} C_n}\right) \tag{19}$$

$$N_{expected\ ,n} = N_{expected\ ,1} \times C_n \tag{20}$$

$$Var\left(N_{expected\ ,n}\right) = N_{expected\ ,n} \times \left(\frac{1 - w}{L}\right) \times \frac{C_n}{\sum_{n-1}^{N} C_n} \tag{21}$$

In candidate site 1, the total observed crashes in a three-year period was 9, and the summation of the correction factor for each year was 3. Thus, the expected average crash frequency was calculated to be 2.42 for year 1, as shown in Eq. (22).

$$N_{expected\ ,1} = 0.31 \times 1.13 + (1 - 0.31) \times \left(\frac{9}{3}\right) = 2.42 \tag{22}$$

Eq. (23) shows the expected average crash frequency for the final year (year 3) in candidate site 1, which is determined using the expected average crash frequency for year 1 and the correction factor for year 3.

$$N_{expected\ ,3} = 2.42 \times 1 = 2.42 \tag{23}$$

Eq. (24) shows the use of expected average crash frequency of 2.42 for year 3 of candidate site 1 to determine the expected crash frequency variance of 1.54.

$$Var\left(N_{expected\ ,3}\right) = 2.42 \times \left(\frac{1 - 0.31}{0.36}\right) \times \frac{1}{3} = 1.54 \tag{24}$$

Table 4-3 shows the predicted crashes of the candidate sites after EB adjustment.

Table 4-3: Crash predictions for the candidate sites after EB adjustment

Candidate Section	Total Predicted Crash	Overdispersion (*k*)	EB Weight (*w*)	Observed	Total Correction Factor	Predicted Crash for the first year	Predicted Crash for the final year	Variance at Final Year
1	3.38	0.66	0.31	9	3	2.42	2.42	1.54
2	0.74	3.93	0.26	3	3	0.81	0.81	3.34
3	10.07	1.04	0.09	14	3	4.55	4.55	6.07
4	17.95	0.46	0.11	10	3	3.62	3.62	2.11
5	0.96	2.62	0.28	8	3	2.00	2.00	5.31
6	12.44	1.61	0.05	6	3	2.10	2.10	4.54
7	0.55	2.46	0.42	15	3	2.96	2.96	5.92

Step 4: Raking of Candidate Sites

Table 4-4 shows the ranking of candidate sites based on the predicted crash number in the final year. Similar procedure can be followed for other facility types (interstates, freeways, arterials, etc.) to obtain the ranking of candidate sites for the application of HFST based on the safety or crash data analysis.

Table 4-4: Ranking of candidate sites

Candidate Section	Predicted Crash for the final year
3	4.55
4	3.62
7	2.96
1	2.42
6	2.10
5	2.00
2	0.81

4.4. BENEFIT-COST ANALYSIS

Once a series of sites has been selected based on crash data and safety analysis, an economic analysis can be performed to determine the benefit-cost ratio. It should be pointed out that money should not be the sole determining factor for the application of HFST. It can be used to aid the decision-making process or to compare multiple proposed locations when funding is limited.

The process to develop the “cost” of a crash is very complicated and can be very sensitive to discuss. Many studies have been conducted to determine such costs for use in benefit-cost calculations. Most studies break down crash costs by severity (fatal, serious injury, injury, property damage only) and take into account both economic and societal factors.

Table 4-5 shows an example of the costs used by MoDOT for such analysis, as provided by MoDOT Highway Safety Division.

Table 4-5: MoDOT estimated crash costs by severity

Crash Type	2022-2026 Cost
Fatal Crash	$11,653,800
Serious Injury Crash	$675,800
Minor Injury Crash	$175,800
Property Damage Only (PDO) Crash	$12,300

As shown in Table 4-5, the costs associated with fatalities and severe or disabling injuries overwhelm the costs associated with minor crashes or property damage crashes. The difference between a fatal or severe injury and a very minor or no injury may be the difference between the occupants' use of available safety equipment, for example, the use of a seat belt since these conditions are beyond the control of an agency, an Equivalent Property Damage Only (EPDO) value is recommended for use. Such costs are developed using the average rate for each severity type and weighted by the probability of their occurrence. Given this methodology, a current estimate of the EPDO is approximately $165,000. If an average cost per square yard of HFST is taken to be $30, it is possible to estimate the cost of a project and compare the expected benefit based on the assumed reduction in crashes. The candidate site no. 3 in Table 4-4 has a length of 0.228 miles. Thus, the approximate HFST construction cost at that location would be around $96,000. It will be shown later in the safety effectiveness section that the application of HFST in two-way two lane rural undivided roadway segment reduces the wet crash by around 65%. Therefore, at that site location, the predicted average wet crash reduction per year is around 3.4. Using an EPDO cost of $165,000, a positive benefit of approximately $467,000 can be calculated for candidate site no 3 in the first year alone.

While the EPDO cost is significantly less compared to the individual crash costs for severe crashes, it is still relatively large compared to the cost of HFSTs. For this reason, benefit-cost can only be used as a ranking method between potential projects, as virtually any project will provide a positive benefit given a reduction of even one crash per year over a ten-year life.

4.5. PAVEMENT CONDITION

Once a decision to proceed with the application of HFST for a segment is made, the next thing to be considered is the physical condition of the roadway. The physical condition will be critical to the successful application and life of the HFST. The data examined in this study did not contain significant variation in condition. Based on Tracker Condition on a scale of 0-10, 125 of the 146 segments were considered "Good." Of the 71 segments on Major Roads, 54 were rated "Good," while 71 of 75 Minor Road segments rated "Good." Similar results were seen when looking at the IRI or Condition Index. In fact, 142 of 146 segments had a Condition Index (0-10) greater than 7.

While MoDOT TMS contains a great source of data for the highway system, it is collected and used generally on a network-level basis. While data is available at intervals of 0.02 miles or about every 105 feet, it should not be used to determine the location of HFST applications without field review.

Many State and local agencies use the PASER system developed by the University of Wisconsin Transportation Information Center to provide guidance on the evaluation of a roadway. It is very straightforward, providing both visual and text descriptions to aid in the consistent rating of pavements. The MoDOT rating for the Condition Index stored in TMS closely follows the PASER rating system. Because of its ease of use in the field and the availability of manuals for both asphalt and concrete pavements, we would recommend its use for the field evaluation of HFST projects.

As discussed previously, HFST is not a structural treatment. It provides virtually no benefit to the distressed pavement with respect to crack filling, rut reduction, etc. Its singular purpose is to increase friction and reduce crashes. Its success is linked critically to its ability to adhere to the pavement surface. This makes the evidence of raveling on the underlying pavement one of the most likely sources of failure. It also highlights the need for strict adherence to the manufacturer's specifications with respect to surface preparation.

Given these criteria, the description provided for a PASER Good condition with a rating of 7 seems an appropriate lower limit for candidate projects with an asphalt surface (Walker, Entine, & Kummer, 2002). The description for PASER rating 7 for asphalt surface is provided in Table 4-6.

Table 4-6: PASER manual rating 7 for asphalt surface (Source: Walker et al., 2002)

PASER Manual Rating	Visible Distress	General Condition/treatment measures
7	• Very slight or no raveling, surface shows some traffic wear. • Longitudinal cracks (open 1⁄4”) due to reflection or paving joints. • Transverse cracks (open 1⁄4”) spaced 10’ or more apart, little or slight crack raveling. • No patching or very few patches in excellent condition.	First signs of aging. Maintain with routine crack filling.

Cracks present in the existing pavement surface must not be structural, i.e., no presence of fatigue, block, longitudinal wheel path, or edge cracking. Although PASER rating 7 accommodates slight raveling, it is not considered acceptable for HFST (NJDOT, 2024; FDOT, 2021). The description of very slight raveling and slight cracking with slight edge raveling should ensure sufficient bond, assuming a clean dry surface at application. Figure 4-11, Figure 4-12, and Figure 4-13 shows the image of asphalt pavement surface with PASER rating of 7.

Figure 4-11: Tight and sealed transverse cracks (Source: Walker et al., 2002)

Figure 4-12: Tight and sealed transverse cracks (Source: Walker et al., 2002)


Figure 4-13: Transverse cracks about 10 feet or more apart (Source: Walker et al., 2002)

For concrete surfaces, a PASER rating 8 is suggested as it accommodates isolated cracks and minor surface defects (Waker, Entine, & Kummer, 2015). Although slight scaling is included in PASER rating 8 for concrete pavements, it should not be allowed for HFST (NJDOT, 2024). The criteria for PASER rating 8 is shown in Table 4-7.

Table 4-7: PASER manual rating 8 for concrete surface (Source: Walker et al., 2015)

PASER Manual Rating	Visible Distress	General Condition/treatment measures
8	• Pop-outs, map cracking, or minor surface defects • Slight surface scaling • Partial loss of joint sealant • Isolated meander cracks, tight or well-sealed • Isolated cracks at manholes, tight or well-sealed	More surface wear or slight defects; little or no maintenance required

Figure 4-14 to Figure 4-17 shows the image of concrete pavement surface with PASER rating of 8.

Figure 4-14: Slight scaling of concrete surface (Source: Walker et al., 2015)

Figure 4-15: Isolated spall at manhole (Source: Walker et al., 2015)

Figure 4-16: Partial loss of joint sealant (Source: Walker et al., 2015)

Figure 4-17: Tight and well-sealed isolated meander cracks (Source: Walker et al., 2015)

In addition to the current condition, the existing pavement age should be considered. MoDOT Planning staff has indicated that the "average" time between overlays for all Missouri pavements is approximately 8 to 10 years. Most overlays are planned to treat pavements before they fall below the Tracker level of "Good," so many of the locations with a PASER rating of 7 or less are probably overlay candidates within 2-3 years. If the safety need is determined to be critical, this may not be a consideration. However, if the need is considered marginal or if the benefit is considered low, consideration may be given to delay the HFST and provide it in conjunction with an upcoming overlay or to move up the overlay to provide maximum HFST benefit.

However, judgment is necessary based on the severity of the crashes occurring at the location. A history of high severity (fatal, disabling injury, etc.) crashes may justify the treatment, or some other countermeasure prior to the new surface being applied.

4.6. EVALUATE SAFETY EFFECTIVENESS

After application of HFST in selected locations, it is important to evaluate their effectiveness. The safety effectiveness of the treatment can be evaluated with or without a comparison group. Safety effectiveness evaluation before and after application of any treatment with Empirical Bayes (EB) does not require any comparison sites. This method of evaluation of the safety effectiveness of the treatment is called EB before and after evaluation. According to the HSM, this method requires at least 10 to 20 sites where treatment has been applied and three to five years of crash and traffic volume data before and after the application of the treatment (AASHTO, 2009).

Once SPF is calibrated for the local conditions for a particular facility type, and some sites have been selected based on the ranking from the crash prediction data, a before and after evaluation of the application of the HFST can be performed to determine its effectiveness. Safety effectiveness was determined for both the dry and wet conditions.

4.6.1. Dry Condition

The following steps can be performed to determine the safety effectiveness of the application of HFST.

1. Applicable and calibrated SPF should be used to determine the predicted average crash frequency for each year in the before period for site type x.

$$N_{Predicted\ -rs} = AADT \times L \times 365 \times 10^{-6} \times e^{-0.312} \times C_i \times AMF_i \tag{25}$$

   Eq. (25) shows the earlier calibrated SPF for the two-lane two-way undivided rural roadway segment. In this study, only the base condition is considered. Thus, the AMFs are taken to be 1. The calibration factor for the local conditions for this type of roadway facility was determined to be 4.86. For HFST section 18 (first row of Table 4-8), crash data was available for six years before the application of the treatment. Using the AADT, length of the section, and calibration factor, the average crash prediction frequency was determined to be 8.4 for the 6-year period before the application of HFST, which is shown in Eq.(26).

$$N_{Predicted\ -rs,18} = 1976 \times 0.36 \times 365 \times 10^{-6} \times e^{-0.312} \times 4.86 \times 6 \times 1.05 \times 1.0 \times 1.16 = 6.8 \tag{26}$$

   It is important to note that Eq. (26) assumes AADT remains the same for the analysis time period.

2. Calculation of EB weight to determine the expected average crash frequency, $N_{expected}$, for the entire before period.

$$w_{i,B} = \frac{1}{1 + k \sum_{Before\ \ Period} N_{predicted}} \tag{27}$$

   Appropriate dispersion parameter should be used for the applicable SPF. The dispersion parameter for the SPF used in this study is provided in Eq. (15). Expected average crash frequency for the before period can be determined using Eq. (28)

$$N_{expected\ ,B} = w_{i,B} N_{predicted\ ,B} + \left(1 - w_{i,B}\right) N_{observ\ ed,B} \tag{28}$$

   For HFST section 18, the predicted average crash frequency for the entire before the period was 6.8. Thus, EB weight was calculated to be 0.18.

$$w_{i,B} = \frac{1}{1 + \frac{0.236}{0.36} \times 6.8} = 0.18 \tag{29}$$

Plugging the value of EB weight, observed crash frequency, and predicted average crash frequency for the entire before period in Eq.(28), the expected average crash frequency for the before period of HFST section 18 was calculated to be 14.3, as shown in Eq.(30).

$$N_{expected\ ,B} = 0.18 \times 6.8 + (1 - 0.18) \times 16 = 14.3 \tag{30}$$

3. Applicable SPF should be used to determine the predicted average crash frequency for the after period in the absence of the treatment. For HFST section 18, three years of crash data was available for the after period. Eq. (23)shows the predicted average crash frequency in the after period for HFST section 18 in the absence of treatment.

$$N_{Predi\ cted\ -rs,18,A} \\ = 1976 \times 0.36 \times 365 \times 10^{-6} \times e^{-0.312} \times 4.86 \times 3 \times 1.05 \times 1.0 \times 1.16 = 3.38 \tag{31}$$

4. The ratio of predicted crashes in the entire after and before period is used to determine the adjustment factor, r, as shown in Eq.(32).

$$r_i = \frac{\sum_{AfterYears} \quad N_{Predicted\ ,A}}{\sum_{BeforeYears} \quad N_{Predicted\ ,B}} \tag{32}$$

For HFST section 18, the predicted crash frequency for the entire after and before period was determined to be 4.2 and 8.4. Thus, using Eq.., the adjustment factor was calculated to be 0.5.

$$r_i = \frac{3.38}{6.8} = 0.5 \tag{33}$$

5. Eq. (34) can be used to determine the expected average crash frequency for each site in the after period in the absence of the treatment.

$$N_{expected,A} = N_{expected,B} \times r_i \tag{34}$$

For HFST section 18, expected average crash frequency in the before period was determined to be 14.30. Thus, the expected average crash in the after period in the absence of treatment is found to be 7.15 using Eq.(34).

$$N_{expecte\ d,A} = 14.30 \times 0.5 = 7.15 \tag{35}$$

6. Safety effectiveness for a particular site can be determined as an odds ratio (OR), as shown in Eq. (36)

$$OR = \frac{N_{observed\ ,A}}{N_{expected\ ,A}} \tag{36}$$

For HFST section 18, the observed crash in the after period was 4. Thus, the odds ratio, according to Eq. (36), was determined to be 0.56.

$$OR = \frac{4}{7.15} = 0.56 \tag{37}$$

7. The safety effectiveness of the treatment can be determined using Eq. (38)

$$AMF = 100 \times (1 - OR) \tag{38}$$

For HFST section 18, the effectiveness of the application of HFST is found to be 44.04%, as shown in Eq. (39)

$$AMF = 100 \times (1 - 0.56) = 44.04\% \tag{39}$$

8. The overall effectiveness of the application of HFST for all the sites in a particular roadway facility can be determined using Eq. (40) to (42).

$$OR' = \frac{\sum_{All\ Sites} N_{observed\ ,A}}{\sum_{All\ Sites} N_{expected\ ,A}} \tag{40}$$

$$OR = \frac{OR'}{1 + \frac{var(\sum_{All\ Sites} N_{expected\ ,A})}{(\sum_{All\ Sites} N_{expected\ ,A})^2}} \tag{41}$$

$$var\left(\sum_{All\ Sites} N_{expected\ ,A}\right) = \sum_{AllSites} \left[r_i^2 \times N_{expect\ ed,B} \times (1 - w_{i,B})\right] \tag{42}$$

For all the sites in the two-way two-lane undivided rural roadways, the total number of observed crashes after the application of HFST was 138, and the total expected average crash without the treatment in the same time period was calculated to be 146.6. Thus, OR' was determined to be 0.94 according to Eq. (40). However, this calculated OR' is potentially biased. The unbiased OR for all the sites considered was calculated using the variance. For HFST section 18, the variance of expected average crash frequency in the after period without the treatment was found to be 2.92, as shown in Eq. (43)

$$variance = 0.5^2 \times 14.30 \times (1 - 0.18) = 2.92 \tag{43}$$

Using Eq. (42), the summation of variances of expected average crash frequency in the after period without the treatment for all the sites was determined to be 89.5. Plugging this number in Eq. (41), the unbiased odds ratio (OR) was calculated to be 0.94 as well.

$$OR = \frac{0.94}{1 + \frac{89.5}{146.6^2}} = 0.94 \tag{44}$$

9. The overall unbiased safety effectiveness of the application of HFST in dry conditions in a two-way two-lane undivided rural roadway can be calculated as a percentage of crash frequency reduction, as shown in Eq. (45).

$$AMF = 100 \times (1 - OR) = 100 \times (1 - 0.94) = 6.28\% \tag{45}$$

The following steps should be followed to determine the statistical significance of the applied safety treatment.

10. The unbiased variance of the safety treatment can be determined using Eq. (46)

$$var(OR) = \frac{(OR')^2 \left[\frac{1}{N_{observed\ ,A}} + \frac{var(\sum_{All\ Sites} N_{expect\ ed,A})}{(\sum_{All\ Sites} N_{expected\ ,A})^2}\right]}{\left[1 + \frac{var(\sum_{All\ Sites} N_{expected\ ,A})}{(\sum_{All\ Sites} N_{expected\ ,A})^2}\right]} \tag{46}$$

The variance for all the sites considered in this study for dry conditions was determined to be 0.01, as shown in Eq. (47).

$$var(OR) = \frac{(0.94)^2 \left[\frac{1}{138} + \frac{89.5}{(146.6)^2}\right]}{\left[1 + \frac{89.5}{(146.6)^2}\right]} = 0.01 \tag{47}$$

11. The standard error of the odds ratio is calculated from the square root of the variance to obtain precision. For HFST in dry conditions, the value was determined to be 0.1, as shown in Eq. (48)

$$SE(OR) = \sqrt{var(OR)} = \sqrt{0.01} = 0.1 \tag{48}$$

12. The standard error of AMF can be determined using Eq. (49)

$$SE(AMF) = 100 \times SE(OR) \tag{49}$$

Application of HFST in dry conditions in two-way two-lane rural undivided roadway segments with horizontal curve yielded a standard error of AMF of 10.03.

$$SE(AMF) = 100 \times 0.1 = 10.03 \qquad (50)$$

13. Statistical significance can be determined from the ratio of Abs [AMF/SE(AMF)]. If the ratio is greater or equal to two, then the application of HFST is effective at a 95% confidence level. The ratio of Abs [6.28/10.03] was calculated to be 0.63 for HFST in dry conditions in two-way two-lane rural undivided roadway segments with horizontal curves.

Table 4-8 and Table 4-9 show the safety effectiveness evaluation and statistical test parameters for the application of HFST in dry conditions in two-way two-lane rural undivided roadway segments.

Table 4-8: Before and after safety effectiveness evaluation in dry condition

HFST Section	AADT	Length (miles)	Observed Crash Before	Time Before Application (years)	Observed Crash After	Time After Application (years)	Predicted Crash in Before Period	Expected Crash in Before Period	Predicted Crash in After Period	Adjustment Factor, *r*	Expected Average Crash in the After Period without Treatment	Safety Effectiveness	Variance Term
18	1976	0.36	16	6	4	3	6.8	14.30	3.38	0.50	7.15	44.04	2.92
31	1091	0.291	3	3	2	2	2.2	2.71	1.46	0.67	1.81	-10.68	0.77
34	1091	0.085	1	1	2	2	0.2	0.51	0.44	2.00	1.03	-94.48	0.78
35	1091	0.123	2	3	1	1	0.8	1.55	0.28	0.33	0.52	-93.44	0.11
50	8569	1.151	16	6	12	4	69.0	19.50	46.03	0.67	13.00	7.69	8.09
56	2981	1.908	18	6	14	3	39.0	21.61	19.52	0.50	10.80	-29.57	4.48
64	1792	0.228	27	6	11	4	20.1	26.69	13.42	0.67	17.79	38.17	11.32
69	4886	0.511	14	5	10	5	29.9	15.07	29.91	1.00	15.07	33.66	14.06
83	688	0.09	9	5	7	2	1.6	7.58	0.64	0.40	3.03	-130.85	0.98
88	4461	0.147	5	6	3	2	24.9	5.49	8.29	0.33	1.83	-64.07	0.59
89	4461	0.173	2	2	2	2	5.4	2.41	5.44	1.00	2.41	16.97	2.12
104	1137	0.096	12	5	2	2	0.9	8.61	0.37	0.40	3.44	41.92	0.96
111	1684	0.321	6	5	1	1	6.4	6.08	1.29	0.20	1.22	17.71	0.20
151	807	0.243	1	1	1	1	0.4	0.54	0.37	1.00	0.54	-84.93	0.14
152	807	0.15	2	3	1	2	1.4	1.81	0.93	0.67	1.21	17.10	0.55
171	1014	0.174	1	2	1	1	0.8	0.92	0.41	0.50	0.46	-118.31	0.12
174	1014	0.17	4	4	4	2	1.8	3.37	0.90	0.50	1.69	-137.20	0.60
175	1014	0.09	3	4	1	1	1.4	2.66	0.35	0.25	0.67	-50.12	0.13
205	2258	0.332	6	5	2	2	6.1	6.01	2.42	0.40	2.40	16.81	0.78
208	2258	0.235	4	3	3	3	2.9	3.70	2.85	1.00	3.70	18.99	2.75
209	2690	0.37	9	3	8	4	4.9	8.01	6.54	1.33	10.68	25.07	10.79
213	3282.5	0.659	13	6	5	3	26.5	14.29	13.26	0.50	7.14	30.01	3.23
214	3282.5	1.307	20	6	13	4	58.2	23.32	38.77	0.67	15.55	16.37	9.46
215	3282.5	0.137	3	4	2	2	4.2	3.15	2.10	0.50	1.57	-27.15	0.69

Table 4-8: continued

HFST Section	AADT	Length (miles)	Observed Crash Before	Time Before Application (years)	Observed Crash After	Time After Application (years)	Predicted Crash in Before Period	Expected Crash in Before Period	Predicted Crash in After Period	Adjustment Factor, *r*	Expected Average Crash in the After Period without Treatment	Safety Effectiveness	Variance Term
217	3282.5	0.081	1	2	3	3	1.1	1.03	1.69	1.50	1.54	-94.29	1.78
252	935	0.349	6	6	3	2	4.7	5.69	1.57	0.33	1.90	-58.10	0.48
258	2412	0.235	6	4	1	1	5.4	5.91	1.35	0.25	1.48	32.32	0.31
260	1216	0.143	1	1	4	2	0.5	0.76	1.08	2.00	1.51	-164.30	1.43
265	2282	0.389	7	5	2	2	6.9	6.98	2.77	0.40	2.79	28.40	0.90
267	2282	0.187	7	5	3	1	4.2	6.55	0.84	0.20	1.31	-128.89	0.22
268	2282	0.115	9	5	2	1	4.1	8.47	0.81	0.20	1.69	-18.05	0.30
278	3492	0.338	10	6	7	5	27.7	10.87	23.08	0.83	9.06	22.72	7.18
279	3492	0.135	1	2	1	1	2.8	1.30	1.38	0.50	0.65	-53.67	0.27

Table 4-9: Test statistics for dry condition

OR'	0.94
OR	0.94
AMF	6.28
Var(OR)	0.01
SE(OR)	0.10
SE(AMF)	10.03
Abs(AMF/SE(AMF))	0.63

4.6.2. Wet Condition

Table 4-10 and Table 4-11 show the safety effectiveness evaluation and statistical test parameters for the application of HFST in wet condition in two-way two-lane rural undivided roadway segments with horizontal curve following the steps outlined above for dry condition. The overall unbiased safety effectiveness of application of HFST in wet condition in two-way two-lane undivided rural roadway was found to be 65.9%. The ratio of Abs [AMF/SE(AMF)] was determined to be 8.97 > 2 for wet conditions.

Based on the analysis, it was observed that the effectiveness of the application of HFST in two-way two-lane undivided rural highways was 6.3% and 65.9% for dry and wet conditions, respectively. The effectiveness of application of HFST was found to be statistically significant in wet condition, but not in dry condition. It is important to mention that the results were obtained using the calibrated SPF with some assumptions regarding the accident modification factors (AMFs) for roadway geometry and design. The obtained results may differ if the site-specific AMFs were considered in the analysis. The results are also expected to vary from one roadway facility type to the next. The example analysis provided in this study is for demonstration purposes only, i.e., how to use crash data to perform safety analysis for ranking candidate sites and evaluate the effectiveness of the application of HFST. It is strongly recommended that SPF be developed for different roadway facilities. If developing an SPF is not feasible, then the SPF available in the literature should be calibrated for the local conditions. Appropriate AMFs should be used with the SPF based on the roadway geometry and design. The developed or calibrated SPF with site-specific AMFs should be used for the safety analysis, i.e., ranking of sites and treatment effectiveness evaluation.

Table 4-10: Before and after safety effectiveness evaluation in wet condition

HFST Section	AADT	Length (miles)	Observed Crash Before	Time Before Application (years)	Observed Crash After	Time After Application (years)	Predicted Crash in Before Period	Expected Crash in Before Period	Predicted Crash in After Period	Adjustment Factor, *r*	Expected Average Crash in the After Period without Treatment	Safety Effectiveness	Variance Term
18	1976	0.36	5	6	3	3	6.8	5.32	3.38	0.50	2.66	-12.71	1.09
19	1925	0.06	1	1	1	1	0.2	0.62	0.25	1.00	0.62	-61.84	0.30
50	8569	1.151	3	6	2	4	69.0	7.36	46.03	0.67	4.90	59.22	3.05
56	2981	1.908	8	6	1	3	39.0	13.33	19.52	0.50	6.66	84.99	2.76
64	1792	0.228	8	6	1	4	20.1	8.56	13.42	0.67	5.70	82.47	3.63
69	4886	0.511	6	5	2	5	29.9	7.61	29.91	1.00	7.61	73.73	7.10
103	1137	0.267	1	1	1	2	0.5	0.65	0.99	2.00	1.30	23.10	0.79
152	807	0.15	1	3	1	2	1.4	1.12	0.93	0.67	0.75	-33.59	0.34
160	807	0.588	4	4	1	1	3.7	3.89	0.93	0.25	0.97	-2.84	0.15
174	1014	0.17	3	4	0	2	1.8	2.66	0.90	0.50	1.33	100.00	0.47
205	2258	0.332	1	5	1	2	6.1	1.95	2.42	0.40	0.78	-28.01	0.25
212	3282.5	0.161	3	4	2	2	4.3	3.18	2.17	0.50	1.59	-25.71	0.69
213	3282.5	0.659	20	6	3	3	26.5	20.62	13.26	0.50	10.31	70.90	4.66
214	3282.5	1.307	13	6	1	4	58.2	16.93	38.77	0.67	11.28	91.14	6.87
215	3282.5	0.137	1	4	1	2	4.2	1.39	2.10	0.50	0.69	-44.02	0.31
252	935	0.349	23	6	1	2	4.7	18.63	1.57	0.33	6.21	83.90	1.58
258	2412	0.235	3	4	1	1	5.4	3.38	1.35	0.25	0.84	-18.50	0.18
278	3492	0.338	13	6	3	5	27.7	13.72	23.08	0.83	11.44	73.77	9.06

Table 4-11: Test statistics for wet condition

OR'	0.34
OR	0.34
AMF	**65.90**
Var(OR)	0.01
SE(OR)	0.07
SE(AMF)	7.34
Abs(AMF/SE(AMF))	8.97

5. CONCLUSIONS AND RECOMMENDATIONS

HFST is a well-accepted countermeasure to reduce roadway departure crashes by improving pavement friction. In appropriate circumstances, it is a cheaper alternative to major safety improvements, such as realignment or geometric correction. Due to its benefits, MoDOT has implemented HFST at many locations across the state. This study reviewed the pavement conditions and crash statistics of these locations before and after the application of HFST to develop guidance or a set of recommendations that can be followed for the successful implementation of HFST at new locations. Based on the findings from this study, the following conclusions can be stated:

- A structured search of the available literature and a focused email survey of 11 state or county representatives was conducted to identify the common early distress in the HFST, the deterioration mechanism, and their effect on the overall performance. The most common distresses observed in the HFST are delamination, cracking, raveling, and surface wrinkling. If the existing pavement has distresses or discontinuities, they propagate to the HFST layer, causing early failure. Thus, many state transportation agencies apply HFST on treated or new surfaces.
- Based on the review of the literature search and survey, crash statistics are the primary criteria for the selection of locations for the application of HFST. However, few have any criteria for determining the suitability of the existing pavement for HFST application. Only Florida and New Jersey have a set of criteria for approving the HFST sites based on the distresses in the existing pavement, such as cracking, rutting, stripping, and raveling.
- Pavement distress data were evaluated before and after the application of HFST to identify locations with early failure. Even at locations with higher reported distress after the application of HFST, few showed the failure of the HFST itself but rather the propagation of underlying distress through the HFST.
- From the crash data analysis, it was observed that there were 925 wet crashes on the 146 sections prior to HFST. This was reduced to 166 in the post-application. However, only 48 of the 146 sections had reduced crashes post-construction, indicating that the large number of wet crashes was concentrated on a relatively small number of sections with large numbers of crashes. Compared to dry conditions, a statistically significant difference in the crashes was observed in wet conditions before and after the application of HFST.

5.1. RECOMMENDATIONS

Using the historical pavement condition and crash data from the HFST sites in Missouri, guidance is proposed for the selection of new locations for the application of HFST.

- Site selection should start with the identification of possible locations based on observed crash data. This can be local knowledge or through examination of network-level data.
- A preliminary safety analysis should be performed to determine the breakdown of crashes by type (run-off road, wet/dry, etc.) and to determine if the location is likely to benefit from the application of HFST. Historical data was also used to demonstrate example calculations relevant to the calibration of SPF, ranking of sites, and safety effectiveness of

HFST on a two-way, two-lane rural undivided roadway segment with a horizontal curve. An average reduction of wet crash frequency was observed to be around 66% after the application of HFST in a two-lane, two-way rural undivided roadway segment with a horizontal curve from the safety effectiveness analysis of the already constructed sites. However, safety effectiveness analysis showed an average crash number reduction of around 7% after the application of HFST in the same roadway segment in dry conditions.

- A benefit-cost analysis can also be performed to determine how effective the application might be. This cost analysis should not be the sole deciding factor on the application of HFST but can be used to rank a series of sites for timing, assuming limited funding is available.
- Once a site is determined to be appropriate, it should be evaluated with respect to the existing pavement condition and if that condition is likely to provide an adequate base for HFST application. MoDOT has indicated that the average service life of treatments on all roadway types is between 8 and 10 years. HFST can be expected to also last 8 to 10 years, so consideration should be given to waiting to apply until routine maintenance is performed to maximize the performance of both. It should be noted that the average age of pavements in this study was 8.8 years at the time of HFST.
- Regardless of age, sites should be evaluated based on current conditions. While ARAN data and video can be used as a screen, the short length of most HFST sites makes a manual survey necessary. The pavement condition component of the Transportation Management System (TMS) database is intended for network-level analysis and reporting. As such, there is no real need for the same rating methods to be employed. A much more usable method, the PASER method, is proposed for use in determining the appropriate conditions for HFST application. A detailed discussion of the method was provided earlier in the report, and a link to the manual is provided in Appendix B. Based on our analysis, it is recommended that the PASER condition of the existing pavement be at least seven or higher for asphalt pavement and eight or higher for concrete pavement prior to the application of HFST.

5.2. OTHER CONSIDERATIONS

- Many actual failures of HFST seem to have occurred on-ramp locations. However, MoDOT does not collect condition data and ARAN images for ramp locations. If this is not available, it is recommended to save the PASER ratings along with photos of pavement conditions before and after the application of the treatment for future analysis.
- Very few HFST sites had friction numbers (FN) before and after the application of HFST. It is recommended that the FN be collected at a regular interval to verify if the HFST is providing adequate benefit even after the observed distresses, which can help define the "failure" of HFST.
- MoDOT may consider a study to develop or calibrate SPFs for different functional classes of pavements based on the local condition and crash data for appropriate safety analysis, which is essential for the successful implementation of the proposed HFST guidance.

Crashes are considered random events, the severity of which can be outside the ability of the agency to control. As such, equations and numbers cannot be the sole determining factor in the application of any safety feature. The proposed guidance on the application of HFST is not exhaustive; rather, it is meant to be revised and updated based on the feedback, suggestions, and engineering judgment of the district and field engineers.

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7. APPENDIX A

AC- Asphalt Concrete
BM- Bituminous Material
HP- Hot In-Place Recycling lace recycling
LC- Leveling Course
PCN- Portland Cement Non-Reinforced
PCR- Portland Cement Reinforced
SM- Stone Mastic AC mix
UTC- Ultrathin Boded Type C
SP – Superpave
ERT- Earth
TP2 – Type 2 aggregate

8. APPENDIX B

Link to flexible pavement PASER manual: Asphalt Pavement PASER Manual
Link to rigid pavement PASER manual: Rigid Pavement PASER Manual