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EVALUATION OF THIN POLYMER OVERLAYS FOR BRIDGE DECKS

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16. Abstract

Polymer overlays consist of polymer resins and aggregates and are used on bridge decks to extend service life by providing waterproofing and chloride penetration resistance and restoring surface friction. High friction surface treatments (HFSTs) are a specific type of polymer overlay intended to provide long-term skid resistance by using aggregates with increased abrasion resistance. This study investigates the performance of HFSTs in Montana to address Montana Department of Transportation concerns regarding their durability and cost-effectiveness. The objective of this research was to assess the factors that influence the long-term friction resistance and durability of polymer overlays, particularly HFSTs, in Montana. The study included a literature review of recent studies on thin polymer overlays and bridge HFSTs, a survey of the experiences of select transportation agencies across North America with polymer overlays, a field investigation in which the condition of HFSTs on select bridge decks across Montana was monitored over a period of three years, and laboratory work to supplement the field investigation and its findings. The study found that the HFSTs investigated are generally of good quality, had adequate installation practices, and have provided at least 5 years of satisfactory performance. The HFSTs demonstrated sufficient bond and excellent electrical resistance after up to 5 years of service and adequate skid resistance after up to 8 years of service. Defects observed in the HFSTs included reflective cracking, often around prior deck patch perimeters, likely due to the use of patch materials with relatively poor thermal compatibility or high shrinkage; reflective transverse cracking in the HFSTs that appeared to increase with time; wear of the HFST particularly in the wheel paths of the driving lanes, in the form of surface aggregate fracturing and loss of aggregate; and minor loss of the HFSTs at the approach joints likely due to snowplow abrasion and traffic impact.

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1. INTRODUCTION

Polymer overlays and high friction surface treatments (HFSTs) applied to bridges are composite materials consisting of organic polymer resins and aggregates. They are primarily used on bridge decks to extend service life, either by restoring surface friction or providing waterproofing and chloride penetration resistance. While polymer overlays generally slow chloride-induced corrosion, the primary purpose of HFSTs, which may be considered a subset of bridge polymer overlays, is to restore and provide long-term skid resistance and as such they use relatively specialized, abrasion resistant aggregates. Polymer concrete overlays are advantageous compared to conventional concrete overlays because they are much faster to install, quick to cure and gain strength, have excellent mechanical and bond strength, and can easily be formulated for a variety of applications. However, the high material cost of the polymer, high thermal coefficient of expansion relative to concrete, moisture sensitivity, safety and flammability considerations, limited resin shelf life, and lack of local contractor experience can be disadvantages. Literature indicates that service lives of up to 25 years or longer can be achieved when the overlay is installed properly, and thicker polymer overlay systems are expected to last longer than thin HFSTs. Thermal-induced fatigue, abrasion or wear from traffic, exposure to ultraviolet radiation, and corrosion propagation and spalling due to the pre-existing conditions of the deck can cause deck or topping deterioration. The overlay performance varies depending on the formulation of the polymer topping, and organic polymer resins used in polymer overlays are typically within the polyester-styrene, epoxy, methacrylate, or urethane families. However, since polymers are routinely formulated and blended with co-polymers, polymer overlays' performance cannot always be generalized according to the polymer's family.

1.1. Problem Statement

The Montana Department of Transportation (MDT) installed high-friction surface treatments (HFSTs) on four bridge decks in 2014 and 2015 in order to improve their skid resistance. An epoxy-based system whose polymer was by Dayton Superior-Unitex was installed on two bridges, one in Bigfork and the other in Big Timber, while a co-polymer system whose polymer was by Poly-Carb was installed on the other two bridges, one in Kalispell and the other in Roundup. The cities' locations are shown in Figure 1. All four HFSTs used Armorstone aggregates supplied by Washington Rock Quarries, Inc. Initial skid numbers were approximately 80 after their construction, but as of 2018 (only 3 or 4 years later), the Bigfork and Kalispell bridges had average skid numbers of approximately 36 and 17, respectively. The Big Timber and Roundup bridges had average skid numbers of approximately 53 and 55, respectively. A skid number of 30 to 35 is typically considered to be the minimum acceptable skid number for highway structures.

The durability of the four HFSTs was called into question by the extremely quick loss of skid resistance at the Kalispell and Bigfork bridges as well as whether or not HFSTs are appropriate across Montana's diverse climates and for Montana's traffic, which commonly uses snow chains in the winter. This study was initiated to develop insight into the long-term performance of HFSTs and determine if HFSTs are an appropriate solution for addressing skid resistance on bridge decks in Montana.

1.2. Project Objectives and Scope

The objective of this research was to assess the factors that influence the long-term performance of polymer-based HFST systems in Montana, specifically with respect to friction resistance and durability, and to provide guidance and recommendations to MDT regarding appropriate polymer systems for use across Montana’s varying geographic regions. In support of this objective, the following tasks were completed:

1. Literature review of the performance reported for HFSTs and thin polymer overlays used in the United States, particularly with respect to friction and durability and in northern states with exposure conditions similar to those encountered in Montana.
2. Survey of select transportation agencies in the United States and Canada that have used HFSTs and thin polymer overlays recently, particularly agencies with geographic regions similar to Montana, to collect information related to installation procedures, the systems used, and their performance.
3. A field investigation in which the condition of HFSTs on select bridge decks was monitored over a period of three years.
4. A laboratory study to assess the durability of the HFSTs observed in the field and characterize their deterioration.

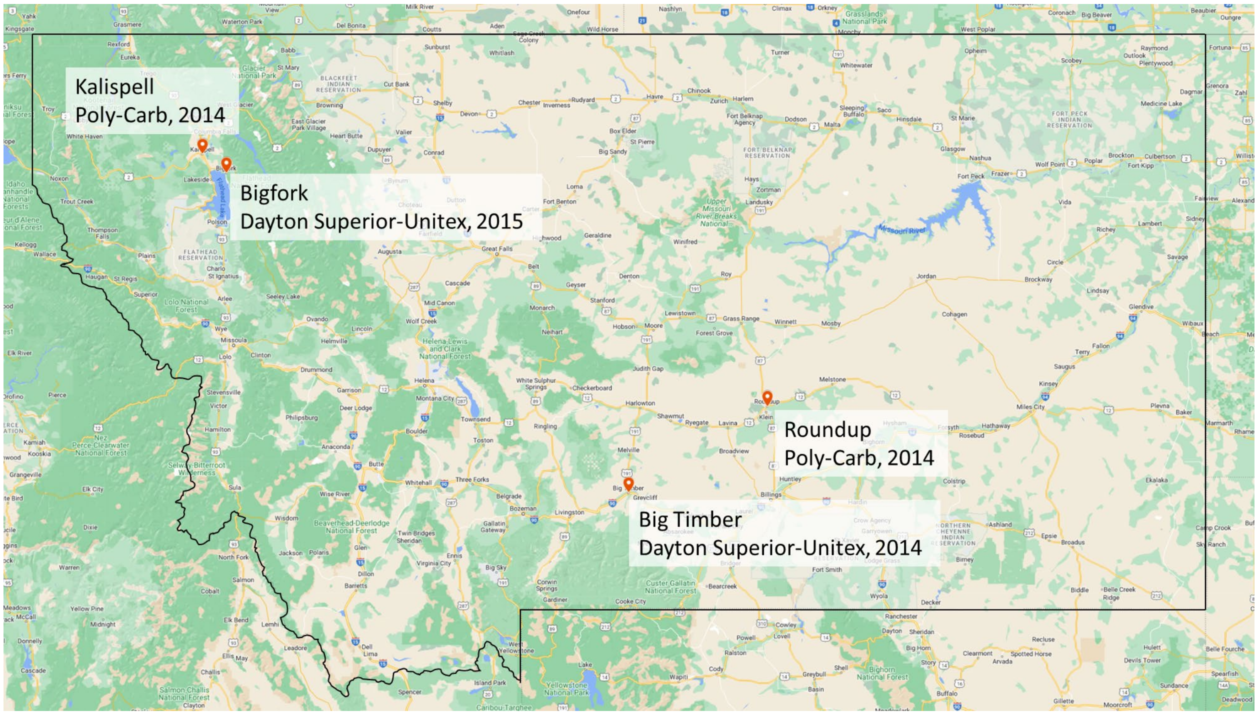

Figure 1. Map showing locations of four bridges in Montana with HFSTs applied in 2014 and 2015.

1.3. Report Organization

This report is organized as follows: Chapter 2 presents the findings of the literature review and Chapter 3 presents a synthesis of the survey responses. Chapter 4 presents the three-year field investigations of the thin polymer overlays and Chapter 5 presents the methods and results of the laboratory studies. Chapter 6 provides discussion of the study results and conclusions. Chapter 7 provides recommendations to the MDT. The survey sent to the transportation agencies and their responses and detailed results of the field investigation and laboratory study are provided in the appendices.

2. BACKGROUND ON POLYMER OVERLAYS AND THEIR USE IN THE UNITED STATES

A literature review of studies conducted by state departments of transportation (DOTs), federal agencies, and other transportation agencies and their experiences with thin polymer overlays was completed to assess the suitability of thin polymer overlays for bridge decks in Montana that require treatments for surface friction or protection. A general overview of when thin polymer overlays are suitable, the materials and techniques used to construct them, and reported performance and degradation mechanisms are compiled. Select studies conducted by northern state DOTs on HFSTs and thin polymer overlays within the last decade are reviewed and the specifications and practices of state DOTs that are expected to have similar exposure conditions to those encountered in Montana are provided.

2.1. General Overview

Polymer overlays have been used somewhat widely across the US since the 1980s and are primarily used on bridge decks to extend their service life by protecting the deck from chemical attack or deicer (chloride) intrusion and subsequent corrosion-induced damage. Polymer overlays focused on protecting the deck from deicer penetration are often applied at thicknesses of about 3/8-inch to 1 inch or thicker. Polymer overlays have also been used to successfully protect decks made with non-air-entrained concrete from scaling and freeze-thaw distress. Adjusting the deck profile and improving ride quality or drainage can be done using thick, screeded polymer overlays but not usually with HFSTs. Polymer overlays are typically not an effective long-term method for mitigating on-going corrosion within heavily chloride-contaminated concrete decks where corrosion has already begun regardless of their thickness since sufficient chloride to promote corrosion remains in the concrete.

High friction surface treatments (HFSTs) have a shorter history of use in the US than polymer overlays, although they are based on similar technology. HFSTs differ from polymer overlays in that their primary purpose is to improve skid resistance and safety, and any protection from chloride intrusion and moisture is considered a secondary benefit. Pavement engineers in the US began adapting the polymer overlay technologies used on bridge decks for increased friction, as HFSTs, in the early 2000s. A HFST on pavement consists of a thin layer of polymer resin, less than 2 millimeters thick, and calcined bauxite aggregates on the order of 1 to 3 millimeters in size (FHWA, 2022). In the context of bridge decks, a HFST is a more ambiguous term and can include multiple layers. A bridge deck HFST may be similar to the systems found on pavements; however in many cases, DOT engineers apply two or more layers on decks to provide better protection from moisture and chlorides than a single layer would, closely resembling broom and seed polymer overlays. A bridge deck HFST often refers to a thin polymer overlay approximately 1/4 inch thick that only differs from a conventional thin polymer overlay (TPO) by the use of special aggregates intended to increase and maintain high friction. Because calcined bauxite is a relatively expensive aggregate, alternative aggregates such as flint, granite, taconite, or basalt are often used, although none have demonstrated resistance to polishing equivalent to that of calcined bauxite (FHWA, 2022). In this report, unless otherwise indicated, a HFST is understood to be a type of thin polymer overlay with specialized aggregates that are relatively hard and have enhanced resistance to abrasion and polishing.

Bridge decks in good to moderate condition, that can only be closed for a short time, cannot handle large increases in dead load, or require minimal modification to joints and drains are good candidates for polymer overlays and HFSTs. Only several hours are required for curing prior to opening to traffic, allowing placement during short, often overnight, lane closures. Bridge decks without significant chloride contamination and corrosion initiation are best suited for treatment as a preventive maintenance strategy. A general guideline is that the deck should have no more than 5% to 10% corrosion-related distress since continued corrosion compromises the service life of the overlay and deck. However, polymer overlays and HFSTs are often still installed on distressed bridge decks to improve skid resistance or to extend the deck service life by slowing the rate of corrosion until rehabilitation or deck replacement can be scheduled. Polymer overlays often can cover and protect static cracks but are not suitable for bridging active (moving) cracks.

2.1.1. Materials

Polymer resins are combined with concrete aggregates to form a polymer concrete. Polymers are versatile and can be formulated with different properties for a wide variety of applications and as such results for one formulation may not be similar to a different material or formulation. Aggregates for HFSTs exposed to tire wear should be angular, hard, and tough (non-brittle) so that they can provide adequate and durable skid resistance without disintegrating under traffic impact. Angular silica, basalts such as trap rock, calcined bauxite, and flint rock are commonly used for this purpose. Several systems have used taconite as well. A Mohs hardness of at least 6 or 7 is preferred by some states and a single-sized or gap-graded aggregate blend is used for HFSTs. Aggregates used in mixer-blended and screeded overlays are fully embedded in polymer concrete and may be smooth and well-graded such that they pack easily, which lowers the required resin content and has both cost and performance benefits.

Epoxy, polyester, and methacrylate are the most commonly used resin binders, although urethanes and blended resins have also been used. Epoxies are the most widespread binder for HFST and polyester-styrene has predominantly been used for mixer-blended polymer concrete that is placed with a screed. The current formulations for polyester polymer concrete overlays were developed by the California DOT in the 1980s and have a high-molecular weight methacrylate primer to repair cracks and improve overlay adhesion. This system has been used successfully by many other northern states. Methacrylates, epoxy-urethanes, and polyurethanes are available, but they have not been as commonly used as epoxies or polyesters.

When selecting a resin/polymer binder, the viscosity, tensile elongation, and modulus of elasticity are considered since these factors affect the performance of the HFST or overlay. Moderate elongation and a moderately low modulus of elasticity of the polymer after curing are generally desirable such that the polymer can accommodate high thermal stresses due to large daily and seasonal temperature changes on exposed decks. A low viscosity may be desired if no primer is used such that the polymer can easily penetrate cracks and wet out the deck substrate; however, low viscosity limits the thickness that can be applied. In addition to these properties, polymers that are resistant to polishing from tire abrasion and resistant to degradation due to acid and alkaline conditions and exposure to ultraviolet radiation are desirable from a durability perspective.

2.1.2. Construction Techniques and Considerations

Polymer overlays may be constructed in one of three ways: (1) the broom-and-seed method, otherwise known as the multiple-layer method; (2) the slurry method; or (3) the premixed and screeded method. In the multiple-layer method, the overlay is placed as though multiple chip seals are being placed on top of each other. Applying a layer of polymer resin to the deck, then broadcasting aggregate on top of the liquid resin, and finally removing unbonded aggregates from the layer once the polymer has cured completes the first layer. This process can be repeated once or twice for a total of 2 or 3 layers and thicknesses typically range from 0.25 to 0.375 inches when using this method. Multi-layer polymer overlays are generally constructed using epoxy as the polymeric binder, but polyesters and methacrylate systems have been used as well.

The slurry method and premixed method are similar in that the polymer resin binder and the aggregates are premixed and then applied to the deck similar to conventional, portland cement concrete overlays. A primer is often necessary to ensure good bond and a layer of aggregates is seeded on the final surface to fill puddles of resin bleed and to maintain initial skid resistance. However, there are several differences between the slurry and premixed methods due to the difference in materials. Polymer overlays constructed by the slurry method typically use methacrylates or low-viscosity epoxies, which include a fine filler but no coarse aggregate. The typical thickness when using the slurry method is about 0.375 inches, although it may range from 0.25 to 0.5 inches. A seal coat may be applied on top of the seeded aggregates to help them bind to the surface. In contrast, premixed polymer overlays typically use polyester resin, although they may use epoxy resin as well, and graded 3/8″ to 1/2″ aggregate. Premixed polymer overlays contain less resin per unit weight than slurries, improving thermal compatibility and reducing shrinkage and the risk of cracking or delamination. They are typically 0.75 to 1.0 inch thick, although thicknesses greater than 1 inch have been successfully constructed as well.

Good surface preparation and understanding and quality control (QC) of resin storage and handling, use of dried aggregate, proper batching and mixing, and overlay placement are necessary for a successful project. Experienced contractors and technical support can be key. The substrate should be sound, the surface should be clean, dry, and free of dust, and any patch repairs should be fully cured and also dry prior to overlay placement. If the substrate is not sound, then it will likely continue to deteriorate under traffic loading and the polymer overlay will not prevent delaminations from continuing to spall. Moisture, dust, asphalts, or oils will compromise the bond between the polymer and the substrate, and between the polymer and aggregates as well. For this reason, aggregates must have a low moisture content (less than 0.2% is desirable), are usually kiln-dried and water-tight bagged, and must also be free of dust and dirt. Reflective cracking in the polymer topping concrete can result if pre-placed deck patches are not fully cured and have substantial shrinkage after the overlay is placed, cracking the patch borders. Polymer resins are required to be stored, batched at proper volumes, and mixed properly or curing and strength can be adversely affected. Pre-job meetings should include discussion of resin and catalyst storage, mixing and placing operations, and avoiding rainy weather. Trial batching and placement can be helpful especially if the contractor is inexperienced.

Ambient site conditions, particularly rain and ambient temperature, can be a challenge for polymer overlay construction. Rain and ponded water make it difficult to maintain a dry surface, which is required for a strong bond. Extreme temperatures affect the viscosity and curing time of the resin, which in turn affect the workability and constructability of the overlay. Unexpectedly cold temperatures may prevent full cure of the polymer while unexpectedly hot temperatures may cause the polymer to cure before the overlay can be finished and to crack under excessive heat. Limiting when construction occurs and proper control of catalyst and accelerator dosages can avoid most problems associated with extreme temperature. Proper surface preparation and QC testing and careful weather monitoring should be done to avoid problems with moisture and rain events.

2.1.3. Performance

Overall, experience has shown that polymer overlays can provide excellent performance when properly designed and applied to sound bridge decks. A typical service life for a properly installed overlay of 10 to 20 years or longer may be expected (Fowler & Whitney, 2011; Krauss, Lawler, & Steiner, 2009), but the life of an overlay may range from 5 to 30 years, or longer, and is typically controlled by proper construction or traffic wear (ElBatanouny, Hawkins, Abdelrahman, Lawler, & Krauss, 2020). Highways where studded tires or snow chains are used can result in particularly high rates of wear and shortened life.

In several studies, such as Tabatabai et al. 2016, polymer overlays have demonstrated superior long-term skid resistance compared to concrete wearing surfaces while in other studies, polymer overlays have not performed as well as concrete surfaces with respect to skid resistance (Soltesz 2010). Depending on the system, polymer overlays can also be relatively impermeable to both chlorides and moisture, particularly if they are maintained.

Shortened service life is experienced when polymer overlays are placed on severely deteriorated and chloride-contaminated decks that have active corrosion. The overlay will reduce the ingress of new moisture into the deck and slow the corrosion rates somewhat, but adequate moisture and chloride remains in the deck and corrosion, delamination, and spalling will continue.

Polymer overlays may perform poorly, either because of shortcomings of the material or flaws in the construction. Materials-related degradation includes the following phenomena:

- Polishing. If the aggregates have poor wear resistance, they and the resin may both become polished, compromising skid resistance. This can be prevented by selecting appropriate aggregates or by using pre-mixed polymer concrete having low resin content.
- Aggregate pop-out. While polymer concrete is relatively impermeable, this property will degrade with time in part because of aggregate pop-out from tire abrasion, impact, and wear. When aggregates are lost during service, they can leave behind fine cracks and small holes through which moisture and chlorides can penetrate (albeit at a slow rate) and reduced skid resistance. Thicker pre-mixed polymer overlays are better able to maintain their impermeability when experiencing surface wear or aggregate pop-outs.
- Thermal incompatibility. Polymers have a higher coefficient of thermal expansion than hydraulic (portland) cement concrete such that they expand and contract more than the concrete substrate during temperature fluctuations. Further, some resins shrink during the polymerization process (curing), which can be additive to shrinkage caused by early-age temperature drops. The concrete substrate restrains the polymer overlay movement, developing stresses that can cause cracking in the overlay and delamination. When polymer concrete is formulated to have low shrinkage and a thermal coefficient relatively similar to that of concrete (or a low resin content), the incompatibility is largely avoided. Generally, low-modulus polymer resin binders that do not develop such high stresses are desirable but lower modulus properties sometimes increase wear and susceptibility to polishing, and can reduce skid resistance.
- Degradation at joints. Polymer overlays tend to experience more degradation and wear at edges and adjacent to joints than in the middle of a span. Increased abrasion from traffic and snowplow damage is common at joint edges. Edges and corners see higher thermal induced stresses as the overlay wants to cup or curl. Other contributing factors may include poor joint maintenance, which permits

moisture and chloride ingress to cause degradation in the underlying concrete which reflects in the overlay, and poor vertical joint alignment during overlay construction.

- Embrittlement. Exposure to sunlight (particularly ultraviolet radiation), heat, and oxygen degrades polymer resins, resulting in embrittlement of the polymer which compromises its ability to accommodate the thermal-induced stresses between the overlay and substrate and can result in resin cracking and delamination. Materials that demonstrate better durability during laboratory testing may be selected, but embrittlement is expected to occur to all polymers. High aggregate loading and topping sand helps reduce the exposure and effects of aging.
- Loss of bond. Bond strength between the overlay and the substrate is important so proper surface preparation must be performed to achieve a sound, clean, and dry substrate. Generally, if good bond strength is achieved during installation, the overlay will perform well but adhesion can decrease with time depending on the overlay compatibility and environmental stressors. Cyclic fatigue and aging can cause the formation of cracks or bond delaminations of some systems, particularly polymers with high modulus (more brittle resins). Polymers with resistance to alkalinity are also needed to maintain bond to the concrete.

Flaws in construction are the more common cause of short-term overlay distress. In addition to inadequate surface preparation or dryness, contractors need to avoid (Fowler & Whitney, 2011):

- Ponded primer,
- Resin-rich areas,
- Non-uniform texture or poor consolidation,
- Inadequate aggregate seeding, and
- Bumps in the surface.

Areas relatively high in resin content, whether the resin is the primer or the polymeric binder, are detrimental because they are more susceptible to shrinkage, thermal-related distress and embrittlement while also having reduced surface friction. Non-uniform texture or light aggregate seeding will cause the surface to polish quickly and bumps in the overlay will experience high traffic wear, particularly impacts from snowplows.

2.2. State Practices

2.2.1. Research Studies

Polymer concretes have been studied and used since the 1950s for numerous applications. Many studies and trials have been conducted on highway and bridge applications for thin polymer overlays and high friction surface treatments since the research topic began in earnest in the 1970s. Initially, broom and seed or slurry-based polymer toppings were investigated. Jenkins, Beecroft, and Guinn (1981) documented the testing program and investigative process used by the Oregon DOT to develop pre-mixed polymer concrete overlays for use within the state. The work was completed from 1973 to 1981 and consisted of laboratory experiments to characterize the constituents and polymer concretes and develop a suitable mix design as well as field applications and evaluations to gain experience in polymer concrete construction. Follow-up inspections of the overlays were conducted to monitor their performance for up to five years. While one overlay debonded after a week, others were in excellent condition at the end of the study after two to five years of service. Based on this experience, the Oregon DOT aided the Idaho

Transportation Department in developing polymer concrete overlays and experience with their construction and hosted a Polymer Materials Seminar in 1979.

Work at the California DOT (Krauss & Neal, 1986) expanded on the work done by the Washington and Oregon DOTs on premixed, polyester polymer concrete. The resin content was reduced by optimizing the aggregate gradation, a silane coupling agent was added to improve resin-aggregate bond, and a high molecular weight methacrylate resin primer was incorporated to improve adhesion and compatibility to the concrete deck.

In 1995, the Washington DOT published a summary of its 10 years of experience with thin polymer overlays (Wilson & Henley, 1995). As of 1995, the Washington DOT had used epoxy and methyl methacrylate (MMA) polymer overlays. Epoxy overlays generally demonstrated greater bond strength than MMA overlays. However, MMA overlays maintained skid resistance longer than epoxy overlays. The MMA overlays had an initial friction number of approximately 40, which decreased to the mid-30s after nine years of service, while the epoxy overlays had an initial friction number of approximately 70, which decreased to the mid- to low 20s in five to seven years. At that time, the Washington DOT noted that latex-modified or microsilica concrete overlays were preferred due to their better durability, unless short traffic closures or minimal deck dead loads were required, in which case thin polymer overlays were suitable.

More recent studies (2010 and later) have been conducted by the Oregon, Minnesota, Colorado, North Dakota, and Wisconsin DOTs. In 2010, the Oregon DOT completed a 3-year study on the performance of thin polymer overlays in Oregon in which the skid resistance and distress of eight different thin polymer overlays were monitored (Soltesz, 2010). The overlays included one with a polyester polymer binder (KwikBond PPC MLS by Kwik Bond Polymers), one with a methyl-methacrylate binder (Safetrack HW, by Stirling Lloyd), one with a urethane binder (Urefast PF60 by LiquidConcrete), and the remaining with epoxy binders (Mark 154 by Polycarb, Flex-O-Lith by Euclid/Tamms, Tyregrip by Ennis/Prismo, SafeLane HDX by Cargill, and Unitex Pro-Poxy Type III DOT by Unitex). The researchers found that none of the overlays performed well under moderate traffic levels; the skid resistance of 7 of the 8 overlays decreased to less than that of the concrete deck control by the end of the three year study. The remaining overlay retained relatively high skid resistance relative to the concrete control, but started to wear through to the concrete after only 2 years. Overall, delamination of the overlays was not a concern.

In 2012, the Minnesota DOT completed a limited survey of the use of ultra-thin polymer concrete overlays (thicknesses between 0.125 and 0.375 inches) on bridge decks (CTC & Associates LLC, 2012). The survey included states from the west (California, Oregon, Washington, Utah and Wyoming), states from the Midwest (Illinois, Missouri, Kansas, Michigan, Wisconsin and Ohio), and from the east coast (New York and Virginia). The majority of states said that they apply polymer overlays when the bridge deck begins to crack or once friction needs to be restored. Utah was unique because at the time of this survey, the Utah DOT used polymer overlays (Pro-Poxy Type III DOT by Unitex and Mark-163 Flexogrid by Poly-Carb, Inc.) on all new bridge decks. Both the Utah and Wyoming DOTs stated that MMA polymer overlays had not been successful, although the Utah DOT noted that this may have been due to poor installation procedures.

In 2014, the Colorado DOT published a study on the performance of thin epoxy overlays on asphalt and concrete bridge deck wearing surfaces (Young, Durham, & Liu, 2014). The study specifically investigated the short-term field performance of the product SafeLane, installed on two decks, and Flexogrid, installed on one deck, over the course of 2 to 3 years. The Flexogrid system experienced local delaminations twice; the reason for the disbondment was not investigated. The SafeLane system experienced reflective cracking on one deck that had an asphalt-overlay-with-waterproofing-membrane that had not been removed prior to installation of the polymer overlay. No major distress was reported for the second deck treated with SafeLane. Both systems demonstrated good skid resistance over the course of the study and effectively prevented further chloride ingress.

The North Dakota DOT conducted a study monitoring the field performance of experimental placements of SafeLane and Flexogrid over 4 years (Loegering & Mastel, 2013). SafeLane was applied over both concrete and asphalt sections and the area installed over asphalt performed relatively poorly. However, instead of reflective cracking, as observed by the Colorado DOT, the asphalt experienced rutting, which caused snow plow damage at areas of the SafeLane overlay that had a relatively high profile. The researchers observed aggregate polishing in the wheel paths of the SafeLane overlays on both the asphalt and concrete sections. They noted that the Flexogrid system was performing well and that the aggregate appeared to have retained most of its angularity. However, some damage at the beginning of the Flexogrid overlay was attributed to impact from traffic and snow plows.

In 2016, the Wisconsin DOT published a study consisting of a limited survey of the Midwest's experience with polymer overlays and a combined laboratory and short-term field study of nine polymer overlay systems (Tabatabai et al., 2016). The system using a two-layer (broom and seed), low-modulus epoxy with flint rock generally had the most satisfactory performance while the system using a 2-layer (broom and seed), polyester-styrene resin with flint rock generally had the poorest performance and fully delaminated during testing. All of the polymer systems exhibited higher friction values than the control concrete section at the end of the study. The group concluded that polymer overlays were suitable for applications when long-term friction enhancements are needed, unless the deck has ongoing corrosion when applied. If the primary purpose of the work is to protect the deck from moisture and chlorides, they recommend using sealers. They additionally noted that during freeze-thaw testing of laboratory slabs, aggregates in the overlays would become loose and that this could be a potential long-term degradation mechanism of polymer overlays.

2.2.2. Specifications

The graphic in Figure 2 shows which early adopter states and provinces reported using thin polymer overlays in the United States and Canada as of 2009 (Krauss, Lawler, & Steiner, 2009). The specifications of Idaho, Wyoming, Utah, Colorado, North Dakota, South Dakota, Michigan, and Alberta were reviewed due to their recent research and publications on polymer overlays and because the region has similar exposure conditions to those found in Montana.

Idaho and Utah have sections dedicated to thin bonded polymer overlays in their most recent Standard Specifications. Idaho specifies a system that uses a high-molecular weight methacrylate (HMWM) primer and pre-mixed polyester polymer concrete overlay. Aggregate within the PPC is to be natural aggregate. Utah specifies a system that uses a penetrating crack filler, followed by an epoxy-urethane overlay. The aggregate is to be basalt, flint, or calcined bauxite.

Wyoming added a supplemental specification to its standard specifications for epoxy-urethane overlays containing silica sand or basalt aggregates. Colorado has a special provision for a polymer system with the same material components as those specified by Idaho (HMWM primer and pre-mixed polyester polymer concrete overlay).

North Dakota, South Dakota, Michigan, and Alberta did not routinely use polymer overlays according to the survey conducted in 2009. However, Alberta has a standard specification for non-skid polymer overlays. The systems incorporated in the specification include epoxy and methyl methacrylate overlays. For the MMA overlay system, basaltic sand and angular silica sand are specified. It should be noted that Alberta has reported good performance of thin polymer overlays in the past and that these overlays are no longer used primarily because of issues reported with the lack of adequate inspection oversight and the wet, rainy climates in some areas of the province that often delay installations.

Michigan published a special provision for thin epoxy polymer overlays in 2016, following a guide on epoxy overlays and healer-sealers for bridge decks. The specification requires a low-modulus epoxy and aggregates consisting of natural silica sand or basalt, or other nonfriable aggregates. In their study, Michigan notes that a Mohs hardness of 7 or more is required in the special provisions rather than 6, as most states require, because the state snow plow blades contain tungsten carbide inserts, which generally have a Mohs hardness of 7 to 7.5 (DeRuyver & Schiefer, 2016).

North Dakota and South Dakota do not have specifications for polymer overlays in their standard specification manuals. However, South Dakota does specify a bridge deck polymer chip seal, which consists of a two-component polymer and aggregates with a minimum Mohs hardness of 6.

Table 1 shows the list of products approved by the Idaho, Utah, and Colorado DOTs. Wyoming and Michigan do not have any polymers for thin bonded polymer overlays listed, but the products they commonly use are shown in Table 2.

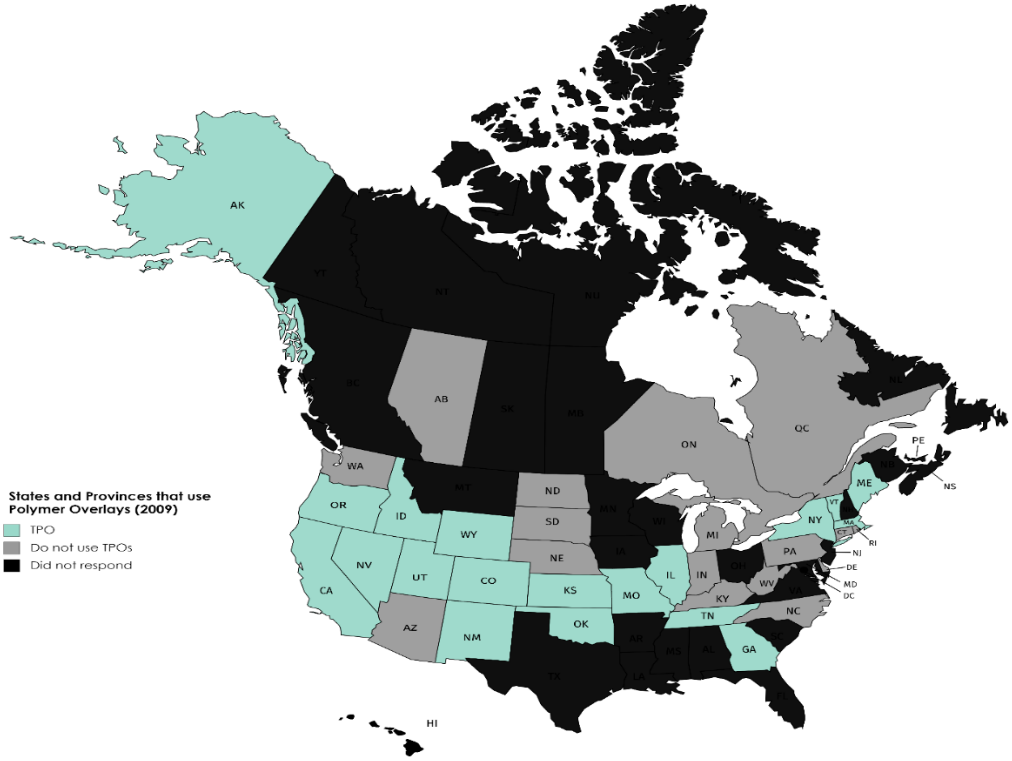

Figure 2. Graphic showing which states use thin polymer overlays (TPOs) according to the survey conducted by Krauss et al. (2009).

Table 1. Products on the Approved or Qualified Products Lists Maintained by ITD, UDOT, and CODOT

Product	**Manufacturer**	**Idaho**	**Utah**	**Colorado**
Flexolith	Euclid Chemical Co.	X		X
Planiseal Traffic Coat	MAPEI Corporation	X		
Planiseal Traffic Coat FS	MAPEI Corporation	X		
Sikadur 22 Lo-Mod	Sika Corporation	X		
Sikadur 22 Lo-Mod FS	Sika Corporation	X		X
Sikadur 25 Lo-Mod EPI	Sika Corporation		X	
Pro-Poxy Type III	Dayton Superior		X	
EPX50-Overlay	E-Chem, LLC			X

Table 2. Products Used by WYDOT and MDOT According to CTC & Associates, LLC (2012)

Product	**Manufacturer**	**Wyoming**	**Michigan**
Pro-Poxy Type III DOT	Unitex	X	X
Mark-163 Flexogrid	Poly-Carb, Inc.	X	X
Flexolith	Euclid Chemical Company		X
E-Bond 526	E-Bond Epoxies, Inc.		X
Mark-154	Poly-Carb, Inc.	X	X
Sikadur 22 Lo-Mod	Sika Corporation	X	X
Transpo T-48 Overlay System	Transpo Industries, Inc.	X	
Akabond 811	Axson		X

Source: (CTC & Associates LLC, 2012)

Other products identified by Tabatabai et al. (2016) include:

- Trafficguard EP35, by BASF,
- ICO Flexi-Coat BD, by International Coating, Inc.,
- PPC MLS, by Kwik Bond,
- PPC 1121 MM MIX, by Kwik Bond, and
- TK 2109, by TK Products.

3. SURVEY OF DOT PRACTICE

A survey on polymer overlays and HFSTs was sent to 16 transportation agencies in order to identify systems that may perform well in Montana's diverse geographic and climatic regions and to identify good practices and procedures for successful systems. Of the 16 agencies contacted (Alberta Transportation, Caltrans, CDOT, ITD (Idaho Transportation Department), MDOT (Michigan DOT), MnDOT, NYSDOT, NCDOT, ND DOT, OregonDOT, PennDOT, SDDOT, UDOT, WSDOT, WisDOT, and WYDOT), 12 agencies responded, as shown in Table 3.

Table 3. List of Transportation Agencies who Responded to the Survey

Responding Agency's Name (Shortened Name)
1. Alberta Ministry of Transportation (Alberta Transportation)
2. California Department of Transportation (Caltrans)
3. Colorado Department of Transportation (CDOT)
4. Michigan Department of Transportation (MDOT)
5. New York State Department of Transportation (NYSDOT)
6. North Carolina Department of Transportation (NCDOT)
7. North Dakota Department of Transportation (ND DOT)
8. Oregon Department of Transportation (OregonDOT)
9. Pennsylvania Department of Transportation (PennDOT)
10. South Dakota Department of Transportation (SDDOT)
11. Utah Department of Transportation (UDOT)
12. Washington Department of Transportation (WSDOT)

The survey included questions regarding the types of materials used and construction and testing requirements, as well as the agencies' experiences with respect to performance and inspection of polymer overlays. While multi-layer, broom-and-seed, thin polymer overlays (TPOs) and HFSTs are of particular interest to MDT at this time, information regarding thicker, mixer-blended and screeded polymer concrete systems, such as polyester polymer concrete (PPC) overlays and epoxy polymer concrete (EPC) overlays, was collected as well. These different systems are defined in this report as:

- **TPOs.** Multi-layer thin polymer overlays are constructed by building up layers to the desired thickness. Each layer consists of mixing and brooming the resin across the deck surface and then broadcasting aggregates over the resin to provide skid resistance. Typically, multi-layer TPOs are two or three layers thick, corresponding to thicknesses of approximately 0.25 to 0.375 inches.
- **HFSTs.** High-friction surface treatments are constructed similarly to multi-layer TPOs. The primary difference is the type of aggregate used. Because HFSTs are primarily intended to increase and provide high levels of skid resistance, very durable and abrasion resistant aggregates are often selected.
- **PPC and EPC Overlays.** Polyester or epoxy polymer concrete overlays are generally applied nominally 0.75 inches thick and may be up to 2 or 3 inches thick in some cases. Unlike multi-layer TPOs, they are

constructed by premixing the resin and aggregate, placing and screeding the polymer concrete mixture, and then broadcasting fine aggregates to fill any resin bleed puddles followed by either tining or grooving the top of the overlay surface.

The following subsections discuss the survey results. Full responses are provided in Appendix A.

3.1. History of Application

Current and historic use of TPOs, HFSTs, PPC overlays, and EPC overlays by the northern state DOTs was surveyed. The use of the various types of polymer overlays discussed in this report by the responding agencies is summarized in Table 4.

Of the responding agencies, OregonDOT, Alberta Transportation, Caltrans, and UDOT were the first agencies to begin using TPOs. OregonDOT began experimenting with multi-layer polymer overlays in 1980 (Jenkins, Beecroft, & Quinn, 1981), and reportedly began using single-lift TPOs comprised of urethanes, methacrylates, and epoxies in the 1980s. The OregonDOT has since switched to using multi-layer, epoxy TPOs. Caltrans and Alberta Transportation began regular use of polymer overlays in about 1985 and UDOT has been using TPOs since before 1990.

MDOT, NYSDOT, and SDDOT began investigating polymer chip seals and TPOs in the 1990s. SDDOT placed its first single-layer epoxy chip seals in 1992 to 1993 and its first multi-layer TPO systems in 2006 to 2007. CDOT reported placing its first epoxy overlays in 2006 and 2007. NYSDOT reports using TPOs for the past 20+ years with good success and, like OregonDOT, NYSDOT limited the accepted polymers for TPOs to epoxies in 2010.

Polyester polymer concrete (PPC) overlays were typically adopted by the agencies later than TPOs, with the exception of OregonDOT and Caltrans. OregonDOT's first experimental polymer overlay installation was a 1.5-inch thick polyester-styrene polymer concrete overlay in 1975 (Jenkins, Beecroft, & Quinn, 1981). Caltrans further developed the polyester-styrene concrete overlays with the first 3/4-inch thick installations occurring in 1983 and 1984. NYSDOT began placing PPC overlays in the 2000s, and began to use them commonly after 2011. CDOT and NCDOT both began using PPC overlays in the 2010s, and NDDOT placed its first polymer overlay, a PPC overlay, in the summer of 2020.

Compared to TPOs and PPC overlays, mixer-blended EPC overlays are considered relatively experimental by the states surveyed. NCDOT placed its first EPC overlay in 2019 and NYSDOT is currently conducting a field performance evaluation of an EPC overlay. The product used by NCDOT and NYSDOT is EPC-Overlay, an epoxy polymer concrete system by E-Chem. The system is installed by premixing the constituents and then placing using a vibratory screed or slip form paver according to its technical datasheet. OregonDOT has completed initial testing and developed a specification for EPC overlays but has not implemented the technology yet; the particular product used in OregonDOT's trials was not identified.

Because HFSTs are similar to TPOs except their primary purpose is to provide skid resistance and more focus is given to the abrasion resistance and durability of the broadcasted aggregates, much of the discussion on TPOs applies also to HFSTs, at least with regard to polymers and surface preparation. Caltrans has used HFSTs for at least 10 years. UDOT reported that while the agency has been installing TPOs for at least 30 years, the agency has only begun using HFSTs within the last 5 years.

Table 4. Summary of Polymer Overlay Use in Responding Agencies

Agency		TPOs[1]	HFSTs[1]	PPC Overlays[1]	EPC Overlays[1]
Alberta Transportation	Current Practice	no longer used[2]	no longer used[2]	--	--
	First Use	1985	--	--	--
Caltrans	Current Practice	in use	in use	in use	--
	First Use	~1985	before 2010	1983	--
CDOT	Current Practice	in use	--	experimental	--
	First Use	2006	--	2015	--
MDOT	Current Practice	in use	in use	--	--
	First Use	1990s	--	--	--
NYSDOT	Current Practice	in use	limited use	in use	experimental
	First Use	1990s	recently	~2006	~2019
NCDOT	Current Practice	in use	--	in use	experimental
	First Use	unknown	--	2016	2019
ND DOT	Current Practice	--	--	experimental	--
	First Use	--	--	2020	--
OregonDOT	Current Practice	in use	--	in use	not yet implemented
	First Use	1980	--	1975	recently
PennDOT	Current Practice	in use	--	in use	--
	First Use	--	--	--	--
SDDOT	Current Practice	in use	in use	--	--
	First Use	2006[3]	--	--	--
UDOT	Current Practice	in use	limited use	--	--
	First Use	Before 1990	2015 to 2020	--	--
WSDOT	Current Practice	no longer used	--	--	--
	First Use	1986	--	--	--

Notes: [1] *"--" means the overlay type was not discussed in the survey response. "Experimental" indicates the overlay type is currently being evaluated under trial application.*

[2] *Alberta Transportation has a standard specification for "non-skid polymer overlays," implying the current system classifies as a TPO and HFST.*

[3] *SDDOT originally experimented with polymer chip seals in 1992-1993, which were single-layer TPOs. The agency placed experimental multi-layer TPOs in 2006-2007 and today uses two-layer TPOs, which they refer to as chip seals.*

3.2. Materials

The types of polymers and aggregates reportedly used by the surveyed agencies are presented in Table 5. Several agencies (MDOT, NYSDOT, OregonDOT, and SDDOT) additionally referenced their Approved or Qualified Products Lists, which are included in Appendix B, for lists of polymer and aggregate sources.

As can be seen in Table 5 and Appendix B, there is a wide selection of polymers available. NCDOT and NYSDOT stated that the type of epoxy TPO selected depends on the average daily traffic (ADT) or the annual average daily traffic (AADT).

Table 5. Polymer Products and Aggregate Sources Used by Survey Respondents, As Reported

Agency	Polymer and/or Manufacturer		Aggregate and/or Source
Alberta Transportation	**TPOs/HFSTs**		Indag #8 Steilacoom 6X10 Bridge Topping
	Flexolith, by Dural Flexogrid, by Polycarb Sikadur 81-32, by Sika Inc Degadur MMA, by Degussa (alternate)		
CDOT	**TPOs/HFSTs**	**PPC Overlays**	No specific sources given
	Unitex Sika epoxy Flexogrid Safelane	KwikBond	
NYSDOT	**TPOs**	**PPC Overlays**	No specific sources given
	E-Bond 526 -Transpo Industries, Inc. New Rochelle, NY EPX50 or EP50-OVERLAY — E-Chem, LLC Albuquerque, NM Flexolith/Flexolith Summer Grade (SG) — The Euclid Chemical Company Cleveland, OH MARK-163 FLEXOGRID — POLY-CARB, Inc. Roberta, GA MasterSeal® 350 — Master Builders Solutions US LLC Shakopee, MN Pro-Poxy Type III DOT — Unitex Chemicals Kansas City, MO Sikadur 22 Lo-Mod FS — Sika Corporation Lyndhurst, NJ SSI RE-DECK — C.S. Behler, Inc. Lancaster, NY	Kwik Bond PPC 1121	
NCDOT	**PPC Overlays**		
	KwikBond Polymers		
OregonDOT	No specific products given		**TPOs**
			Armorstone, from Washington Rock Quarries Traction Control, from Earth Work Solutions
PennDOT	KwikBond polyester polymer concrete		No specific sources given
SDDOT	Transpo T48, by Transpo Ind, Inc Polycarb Mark 163		No specific sources given
UDOT	**TPOs**	**HFSTs**	No specific sources given

Agency	Polymer and/or Manufacturer		Aggregate and/or Source
	Dayton Superior E-Chem Sika	KwikBond	

3.3. Guidelines for Application

Many of the responding agencies identified situations for which polymer overlays would be considered appropriate and situations for which they would be considered inappropriate. While there were some common trends, practices varied between the agencies.

The general sentiment is that polymer overlays can provide a skid-resistant surface while sealing the deck from moisture and chloride intrusion; TPOs and PPC overlays are expected to provide chloride and moisture protection by most agencies. Two agencies are currently evaluating the chloride protection offered by PPC overlays and the protection offered by premixed EPC overlays has not yet been reported due to their experimental status.

Alberta Transportation stated that polymer overlays were originally considered because they do not add much dead load to the structure, do not shorten curb or barrier heights, and were considered a cost-effective method of sealing cracks.

Regarding HFSTs, Caltrans does not consider HFSTs to be effective against chloride penetration, and as a result does not use them for deck protection purposes. However, Alberta Transportation, whose TPOs are designed as “non-skid” polymer overlays, consider them to be an effective protective system against chlorides, provided conditions of the existing deck are favorable.

Caltrans does consider TPOs/HFSTs to be an effective surface treatment for skid resistance, and Caltrans identified a special niche for thin polyester HFST as a sacrificial layer to protect thicker PPC overlays from high traffic and abrasive wear. MDOT applies TPOs on concrete decks as a sacrificial layer to protect the deck from wear due to snowmobile treads and reapplies the TPO once it is worn, typically after about 5 years. In acknowledgement of a TPO’s limited life under high traffic, NCDOT chooses between TPOs and thicker PPC overlays based on the average daily traffic (ADT) and truck traffic experienced by the deck.

According to NYSDOT, NCDOT, and WSDOT, polymer overlays should not be used on deteriorated decks; NYSDOT in particular noted that while thicker PPC overlays are more durable than TPOs, these thicker overlays are still ill-suited for deteriorated decks. NYSDOT also stressed the importance of applying the polymer overlay prior to deck deterioration in order to achieve an effective surface for overlay adhesion and to result in a deck with minimal maintenance needs. NCDOT stated that polymer overlays typically are not used on bridge decks with a general National Bridge Inspection (NBI) rating less than 6, with some exceptions. Furthermore, NCDOT does not permit polymer overlays to be used if the deck has heavy chloride contamination, defined as 2 pounds per cubic yard near the top mat of rebar, despite the fact that this precedes any chloride-induced corrosion-related deck distress. Conversely, OregonDOT typically includes polymer overlays, particularly PPC overlays, as part of their deck rehabilitation strategy, which also consists of crack repair and concrete patching to address delaminations and spalls, although TPOs are more often applied as a preservation measure when the deck is in good condition.

Other scenarios for which polymer overlays are not considered appropriate include:

- If a previous overlay is present and cannot be removed, in which case many contractors and material suppliers will not place a PPC overlay (PennDOT); or
- If the deck belongs to a movable bridge (WSDOT).

While TPOs are typically considered to be a good choice for preservative maintenance due to their ability to seal decks and cracks long-term and their relatively low initial cost compared to other overlays, Alberta Transportation warns that TPOs are not always the most beneficial option due to frequent premature failures and costs of long-term maintenance.

PennDOT notes that PPC overlays are an alternative to latex-modified concrete (LMC) overlays or, in the case of one district, hot-mixed asphalt (HMA) overlays with a waterproofing membrane. These comments serve as a reminder that while polymer overlays may be well-suited for a particular deck, they may not be the most cost-effective strategy and a life cycle cost analysis (LCCA) may still be warranted to compare between TPOs, polymer (PPC) overlays and alternative systems.

3.4. Performance

Agencies were asked to identify how long their polymer overlays have been in service and common causes of failure. Additionally, agencies were asked if they had evaluated or monitored the chloride penetration resistance and skid resistance of polymer overlays quantitatively.

3.4.1. Service Life

3.4.1.1. Thin Polymer Overlays and High Friction Surface Treatments

Experience-based service life estimates for TPOs and HFSTs are shown in Figure 3. Estimates between 5 and 15 years were most common, but the expected service life varied from as little as 2 years, as reported by Caltrans for areas with very high amounts of traffic and snowfall (tire chain wear), to as long as 35 years and counting, according to Alberta Transportation. The life of the overlay depends on the aggregate type, AADT, and structure geometry according to NYSDOT, and on the traffic, winter weather, and number of snowplow passes according to SDDOT. MDOT noted that TPOs can wear out in 5 years when exposed to snowmobile treads.

NYSDOT and UDOT stated that they expected service lives of HFSTs to be 10 to 15 years and 15 years, respectively. However, because HFSTs are still experimental in these states, the actual service life is still being verified.

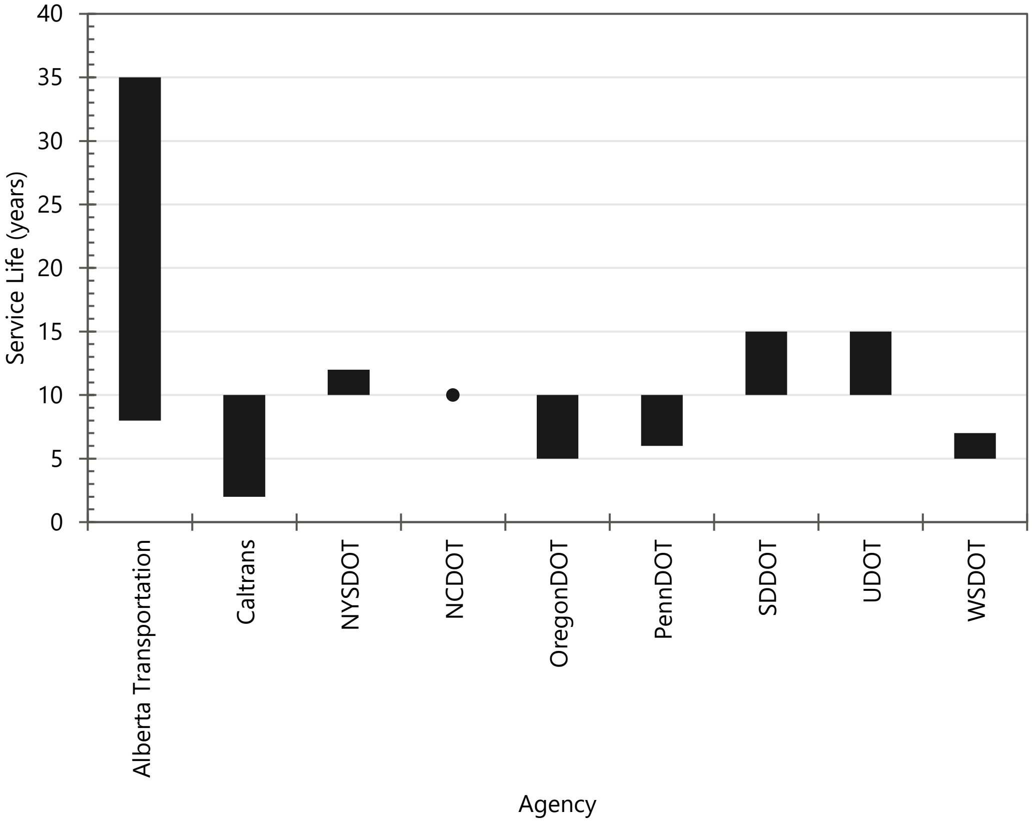

Figure 3. Reported ranges of service life for thin polymer overlays and high friction surface treatments. (PPC overlays are discussed in the next subsection.)

3.4.1.2. Polyester Polymer Concrete Overlays (PPC)

OregonDOT stated that PPC overlays have demonstrated 15 to 20 years of service life. Caltrans first installed PPC overlays in about 1984 on bridges with low ADT, and while they have worn though in spots, they still remain generally functional after over 30 years. PPC overlays on mountainous roads subjected to winter tire chains have had shorter service life of 10 years or less. PennDOT, NCDOT, and NYSDOT all anticipate long service lives of 20 to 30 years. However, these states have begun using PPC overlays relatively recently and the service lives have not been validated yet. The oldest PPC overlay discussed within these states is a 14-year-old overlay that is still performing well according to NYSDOT.

3.4.2. Causes of Failure

Common causes of failure for TPOs, HFSTs, and PPC overlays include debonding/delamination, cracking, wear or abrasion, and material degradation.

- **Debonding/Delamination.** Of the 12 survey respondents, 8 respondents cited debonding and delaminations as the failure mode and the cause of the failure varied from early in the life of the overlay due to poor construction quality to late in life due to aging and exposure. Alberta Transportation, CDOT, MDOT, NCDOT, PennDOT, and SDDOT identified the following as causes of premature (early-age) failure:
    - Improper surface preparation,

  - Poor epoxy resin mixing,
  - Inadequate curing,
  - Improper use of primer,
  - Inadequate QC provisions,
  - Calibration errors in equipment, and
  - Inappropriate epoxy/resin storage.

  Caltrans, PennDOT, and WSDOT noted that delamination is the most common failure if traffic abrasion and surface loss does not control service life. SDDOT estimated that approximately 10% of overlay area fails at the bond and is currently investigating the cause(s). Alberta Transportation stated that debonding may occur due to differences in the coefficient of thermal expansion (COTE) of the polymer overlay and concrete substrate and thermal cycling (fatigue) may promote delamination. NYSDOT was the only state to report that failure by delamination is relatively rare.

- **Cracking.** Cracking of the overlay was cited by only 3 survey respondents. NYSDOT stated that reflective cracking due to movement of the structure or underlying active cracks is the most common cause of cracking. UDOT also identified reflective cracking as a concern. Cracking is also an early-age failure mechanism according to Alberta Transportation that may be caused by a poor batch of material or joint movement during installation or curing.
- **Wear/Abrasion.** Of the 12 survey respondents, 6 respondents discussed failures related to wear and abrasion, which typically makes the skid resistance of the surface inadequate. WSDOT attributed loss of friction to loss of aggregates and noted that aggregates are mostly pulled out by studded tires while UDOT attributed loss of friction to aggregate polishing, particularly polishing of aggregates with flint. NCDOT indicated that skid number controls life in high-traffic areas while CDOT and MDOT attributed heavy damage to chains on trucks in mountain passes and snowplow damage, respectively. Alberta Transportation has observed that once the riding surface becomes rough, rainwater becomes trapped in low spots and slowly propagates failure. OregonDOT noted polymer overlays in high elevation snow zones where studded tires are used experience increased rutting.
- **Material Degradation.** Alberta Transportation was the only agency to identify a material degradation mechanism (hardening or embrittlement of the polymer binder due to ultraviolet radiation) as a common failure mode.
- **Miscellaneous.** Other causes of failure mentioned by the respondents include:
  - Inadequate aggregate coverage, particularly due to heavy tining of the concrete surface (SDDOT); and
  - Failures at deck joints due to insufficient identification of joint repair needs (NCDOT).

CDOT and PennDOT additionally discussed the effect of climate. CDOT noted that the agency has experienced curing issues in severe winter climates, while PennDOT stated that severe winter climate does not seem to impact the service of the overlay as long as the bond is of good quality.

Table 6. Proposed or Standard Mitigation Strategies to Prevent Polymer Overlay Failure

Respondent	Cause of Failure/Distress	Mitigation Strategy	Status
CDOT	Equipment calibration errors	QA bond strength testing	Standard practice
Caltrans	Overlay delamination of PPC	Use of a methacrylate primer Increasing deck concrete profile[1]	Standard practice Under investigation
UDOT	Reflective cracking (PPC)	Use of a healer/sealer primer	Under investigation
UDOT	Aggregate polishing HFST	Prohibiting use of flint rock	Standard practice
SDDOT	Inadequate aggregate coverage	Requiring diamond grinding	Standard practice

Notes: [1] *As discussed later, MDOT is also currently investigating the effect of increasing the deck concrete profile.*

3.4.3. Chloride Intrusion and Skid Resistance

Of the agencies surveyed, Alberta Transportation, Caltrans, and CDOT have quantitatively evaluated polymer overlays' ability to seal concrete decks from chloride intrusion. CDOT published a study in 2014 (Young et al., 2014) in which chloride profiles of several bridge decks were tested before and up to 4 years after overlay installation, and the data demonstrates that the overlays effectively prevented chloride intrusion. The overlay systems evaluated were SafeLane, by Cargill, which uses Unitex Pro-Poxy Type III DOT resin, and Flexogrid, by PolyCarb. Alberta Transportation also concluded that overlays effectively prevent chloride intrusion, but added that performance depends on the condition of the existing overlay, if one is present, crack frequency, the presence of pre-existing chlorides in the concrete substrate, and severity of the deicer application practices. In contrast, Caltrans evaluated a portion of delaminated overlay and noted that the porosity was visible, concluding that TPOs cannot be relied on for sealing decks from deicers.

While the other agencies have not completed quantitative analyses, the general consensus based on experience and field observations is that polymer overlays, specifically TPOs and PPC overlays, are effective in keeping out chlorides if they remain well bonded and intact. Alberta Transportation stated that a thin polymer overlay can add at least 20 years to a bridge deck's life and MDOT has observed minimal delaminations of the underlying deck when replacing epoxy overlays after 15 to 20 years of service.

Quantitative evaluation of skid resistance is more common than that of chloride intrusion. Skid trailer tests are commonly used to evaluate skid resistance and several state DOTs follow ASTM E274, *Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire*. Alberta Transportation and NYSDOT reported monitoring skid resistance of select bridges with TPOs, HFSTs, and/or PPC overlays regularly and UDOT monitors skid resistance of state routes annually and evaluates specific structures on request. Caltrans assesses skid resistance of the as-built condition of the overlay but does not monitor friction quantitatively. CDOT, MDOT, OregonDOT, SDDOT, and WSDOT have assessed skid resistance of polymer overlays or their aggregates in research projects.

The skid resistance results are mixed. Alberta Transportation and CDOT have found that polymer overlays do perform better than typical highway pavements or deck concrete mixtures. In the 2014 study by Young et al., CDOT observed that thin polymer overlays initially provide higher skid resistance values than the concrete surface and while the initial increase is lost after about one year, further decreases in skid resistance are less dramatic and occur at a decreasing rate with time. The overlays retained satisfactory skid resistance throughout the study, although monitoring was only completed for a few years after overlay installation.

In comparison, NYSDOT and UDOT stated that results are dependent on the aggregates used, contractor experience, and placement method, and Caltrans and NCDOT indicated that skid resistance does not typically control overlay life. NYSDOT noted that calcined bauxite tends to retain skid resistance well.

WSDOT and OregonDOT have concluded that some polymer overlays do not perform very well with regards to skid resistance. The OregonDOT conducted a study in 2010 (Soltesz, 2010) and found that the skid resistance of even the best-performing overlay systems assessed was expected to be equivalent to that of a concrete surface within 5 months with a traffic load of 10,000 ADT per lane.

3.5. Material Properties

Most of the survey respondents identified requirements for aggregate properties instead of listing specific sources. The most common types of requirements are:

- **Aggregate Composition.** For skid resistance, state DOTs primarily discussed flint, basalt, and calcined bauxite aggregates. While some agencies such as CDOT permit flint aggregates, others including PennDOT do not permit flint aggregates, which tend to polish according to UDOT. Basalt is common and calcined bauxite is often used in HFSTs. The aggregates premixed within polyester polymer concrete overlays are typically siliceous rounded gravels that are dried and prebagged.
- **Gradation.** Three respondents (MDOT, ND DOT, and OregonDOT) reported having gradation requirements for aggregates used in TPOs and HFSTs, PPC overlays, and TPOs, respectively. MDOT requires that the aggregate gradation meet Table 2.3 of ACI 548.8-07, *Construction Spec for Type EM (Epoxy Multi-Layer) Polymer Overlay for Bridge and Parking Garage Decks*, which is shown in Table 7.
- **Moh's Hardness.** The minimum hardness specified ranges from 6 to 7. MDOT and ND DOT both specify a hardness of at least 7. PennDOT requires a hardness of at least 7 for PPC overlays but will accept a hardness of at least 6.5 for epoxy overlays, and OregonDOT requires a minimum hardness of 6 for multi-layer polymer overlays.

Table 7. Gradation Requirements of ACI 548.8-07

Sieve Size (mm)	Percent Passing
4.75	100
2.36	30 to 75
1.18	0 to 5
0.600	0 to 1

Source: ACI 548.8-07, Construction Spec for Type EM (Epoxy Multi-Layer) Polymer Overlay for Bridge and Parking Garage Decks

For TPOs (HFSTs), OregonDOT further has requirements for aggregate absorption, abrasion loss, and fracture quantities. A maximum absorption of 1.25% is permitted as measured by AASHTO T 84, *Standard Method of Test for Specific Gravity and Absorption of Fine Aggregate*, and a maximum abrasion loss of 2.8% is permitted as measured by a modified version of ASTM D7428, *Standard Test Method for Resistance of Fine Aggregate to Degradation by Abrasion in the Micro-Deval Apparatus*.

Alberta Transportation selects aggregate sources that have the best performance of the sources available in the area. OregonDOT also noted that the overall performance of the aggregates used has been good, either despite or because of their variability. SDDOT has found that basalt aggregates tend to be less dusty than quartzite and observed that darker aggregates perform better in winter. Caltrans noted that all the aggregates are susceptible to snowplows, and that copper slag aggregates polish very quickly.

3.6. Construction Specifications

When asked how they prepare the deck surface for polymer overlays and the QA/QC procedures used to ensure adequate preparation and successful overlay performance, many survey respondents identified the applicable standard specifications or special provisions for polymer overlays developed by their agencies and these have been compiled in Appendix B with the approved products lists identified previously. The general surface preparation procedures consist of:

1. **Milling or scarifying the deck surface.** Milling or scarifying is done to remove chloride-contaminated concrete and existing overlays. Because it removes concrete cover, it is typically only conducted when applying thicker PPC overlays, which can recover the concrete cover and maintain the deck elevation. While milling produces a rough surface, NCDOT and others still require shotblasting or other abrasive blasting in order to achieve a sufficient profile, micro texture, cleanliness, and bond strength for polymer concrete.
2. **Washing and drying the deck surface.** PennDOT was the only respondent to identify washing the deck with a degreaser and then drying the deck with compressed air prior to shotblasting. Often, shotblasting is the primary method specified for removing surface contaminants.
3. **Shotblasting the deck surface.** Steel shotblasting is conducted in order to clean the deck surface and produce the specified deck surface profile and cleanliness. Several respondents noted that their agencies require sandblasting in areas inaccessible to shotblasting. Contaminants that must be removed include concrete laitance (such as weak surface mortar or loose or softened concrete), asphaltic materials (such as membranes or asphaltic concrete), coatings, oil, grease, slurry, paint, dirt, striping, curing compound, and rust. In general, the surface profile must expose the coarse aggregate in the substrate. Concrete surface profiles (CSPs) of 5, as defined by the International Concrete Repair Institute (ICRI), are commonly required by manufacturers and NYSDOT typically requires a CSP of 5 or 6 for TPOs and HFSTs. The respondent from MDOT noted that MDOT specifies a CSP of 7 such that no areas of the deck have a profile less than CSP 5 and that the higher profile enhances the bond area.

In order to ensure the polymer overlay is of good quality, the agencies require QA/QC testing of the materials, the prepared surface, and the finished overlay. Alberta Transportation, OregonDOT, and PennDOT require material testing of the polymer, aggregates, and/or the overlay material. For example, Alberta Transportation requires infrared and gas chromatography analysis of each polymer component, grain size analysis of the aggregate, and compressive strength and modulus of elasticity (MOE) of the polymer mortar. The NCDOT respondent did not identify any material testing but stated that the batching process and batch tickets are monitored.

After shotblasting is complete, the surface profile, moisture, and cleanliness of the deck are all verified. Moisture testing is commonly conducted per ASTM D4263, *Standard Test Method for Indicating Moisture in Concrete by the Plastic Sheet Method*, or the moisture content may be measured with a moisture meter. The maximum moisture content permitted by NYSDOT is 5%. NCDOT requires a visual inspection of the deck after shotblasting to verify that the substrate is sound, and a sounding survey is conducted if there are any questionable areas. Ambient conditions must also be suitable for overlay installation to progress. NCDOT requires a concrete surface temperature between 40°F and 100°F and OregonDOT specifies a wait time after rain events or mechanical heating of the deck in order to achieve a dry surface.

The survey respondents identified the following post-construction acceptance testing requirements:

- Bond testing per ASTM C1583, *Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-Off Method)*;
- Skid testing per AASHTO T 242, *Standard Test Method for Frictional Properties of Paved Surfaces Using a Full-Scale Tire*;
- Smoothness quality testing; and
- Compression testing of cast samples.

The ND DOT also requires a sounding survey after construction is complete to identify any areas requiring remediation. The respondents did not identify the acceptance criteria, except for the NYSDOT respondent who stated that the minimum permissible bond strength for TPOs is 250 psi and the minimum friction number is 65.

Finally, Caltrans, NCDOT, and NYSDOT emphasized staff qualifications and oversight. NYSDOT requires a competent manufacturer representative be present during surface preparation, material placement, and any remedial work completed. NCDOT further requires the overlay supplier and its technical representative and the contractor to hold qualifications. Caltrans stated that a knowledgeable State representative should be on site during all operations.

3.7. Inspection of Bridge Decks with Polymer Overlays

Reportedly, the following methods are commonly used in routine inspections of bridges with polymer toppings or overlays:

- Visual inspection of the top surface for cracks, spalls, patches, debonding, and other defects,
- Visual inspection of the soffit (if the superstructure permits), and
- Sounding of the top surface by chain drag or hammer methods to identify debonding or delamination limits.

However, responses regarding the effectiveness of visual inspection and sounding were mixed. Several agencies including Alberta Transportation and Caltrans noted that visual inspection of the concrete deck surface is limited by the overlay while OregonDOT noted that the presence of a polymer overlay does not affect their inspection procedures and PennDOT also stated that visual inspection is not impacted if TPOs are present because the deck defects reflect through the overlay. SDDOT commonly conducts a thorough and detailed deck inspection prior to applying the overlay since it is difficult to get an accurate understanding of the existing deck condition once the overlay is in place.

Hammer sounding and chain dragging are most commonly used to locate delaminations. It is generally accepted that with sounding it is difficult to distinguish between delaminations at corroded rebar and debonded overlay, but MDOT stated that inspectors in their state are typically able to tell the difference due to their long history of polymer overlay use and subsequent experience. PennDOT additionally noted that sounding is not effective if the material has rebound properties, which is common for thicker PPC. Several agencies have used infrared thermography and spectral wave analysis technology to identify the boundaries and depth of deck deterioration.

There is also some disagreement on the usefulness of ground-penetrating radar (GPR). CDOT uses GPR to measure overlay thickness and identify rebar locations while PennDOT does not use GPR. Other methods that are not commonly used include active corrosion monitoring, linear polarization, and half-cell potential surveys since the polymer overlay provides electrical insulation.

Destructive test methods include bond testing, coring, and chloride testing. Because of their destructive nature, they are typically used relatively sparingly on projects. For example, NYSDOT only permits bond testing when distress is visible and NCDOT conducts chloride testing only when a repair or preservation activity is under consideration. However, Alberta Transportation is an exception and conducts chloride testing according to "Method for Field Determination of Total Chloride Content" as described in SHRP-S-328 (Herald et al., 1993) and "Method for Chloride Content in Concrete Using the Specific Ion Probe" as described in SHRP-S-330 Appendix F (Cade & Gannon, 1993) regularly.

3.8. Maintenance of Polymer Overlays

Agencies believe that polymer overlays cannot be regarded as a "one-and-done" activity and require maintenance themselves. The survey respondents expected the following actions to be necessary during the life of the overlay:

1. **Chip Seal/Additional Layer/Reapplication.** Regular application of an additional layer, or reapplication if the overlay is in sufficiently poor condition and when the deck substrate remains in good condition, may be required approximately every 10 years. A new layer will restore friction, rideability, and chloride and moisture impermeability, and can address minor defects in the overlay, thereby extending the serviceable life of the overlay and the protection of the deck.
2. **Patching of Spalls and Debonded Areas.** Alberta Transportation indicated that the coefficient of thermal expansion (COTE) of repair materials and material selection needs to be carefully considered when patching a deck prior to placing a polymer topping or overlay. Epoxy mortar patches have often failed at the bond line because of differences in COTE between the repair material and the substrate materials. Failed patches results in cracking and sometimes spalling of the overlay.
3. **Rut Maintenance.** If heavy traffic results in rutting of the polymer overlay, the ruts may be filled with repair materials. OregonDOT has successfully used epoxy and PPC to fill ruts in polymer overlays, decks, and roadways.
4. **Crack Sealing.** NYSDOT identified cracking as the most common defect in their overlays and recommends sealing thin cracks in TPOs and HFSTs with the same resin binder. For PPC overlays, NYSDOT seals cracks with the same high molecular weight methacrylate (HMWM) primer used when installing the overlay and then broadcasts dry sand on top.

3.9. Other Practices and Insight

The agencies were specifically asked about the materials they had used in polymer overlays, their construction procedures and specifications, the performance of the overlays, and how polymer overlays affect inspection of the bridge deck. Based on their experiences, the survey respondents additionally identified or recommended the following practices:

- **Monitor Polymer Overlays.** A polymer overlay may not always be the most cost-effective preventive maintenance and a life-cycle cost analysis (LCCA) may be advisable to confirm its suitability. However, an accurate analysis depends on an accurate understanding of how much maintenance the overlay requires, which is often underestimated. Alberta Transportation and CDOT recommended monitoring polymer overlays and reviewing the performance of previous projects in order to develop accurate life cycle costs (LCCs). LCCs may be minimized by installing the overlay while the substrate is in good condition and ensuring the substrate is properly prepared, which can result in almost no maintenance needs according to NYSDOT. PennDOT has maintenance crews that can apply epoxy TPOs which has provided savings on initial costs and is developing the capability to perform PPC overlay work with department forces.
- **Implement Trials.** CDOT and PennDOT require trial applications or test strips to be completed prior to full installation on the structure. PennDOT may wave this requirement if the contractor has significant experience and satisfactory history. In their recent overlay project, ND DOT specified that the contractor had to have installed a PPC overlay within the last 5 years and additionally required a trial.
- **Require a Warranty.** Alberta Transportation, MDOT, and PennDOT use warranties to help ensure polymer overlays are of good quality. Alberta Transportation and MDOT both require a 5-year warranty on TPOs and HFSTs. MDOT requires 5 years since in their experience, failures due to improper installation typically occur within 2 or 3 years. PennDOT has also begun implementing contracts wherein the contractor is responsible for the maintenance of the bridge for 25 years, in which no major element (deck, superstructure, or substructure) may have a National Bridge Inspection (NBI) rating less than 6. In response, contractors have begun applying PPC overlays in order to minimize maintenance costs during this time.

  PennDOT recommends ensuring the warranty is supported by the material manufacturer. The warranty adopted by Alberta Transportation is a joint manufacturer-contractor warranty.
- **Best Practices.** Miscellaneous practices implemented by the surveyed agencies that have successfully improved performance include:
  - Using tining and strict control of resin content in PPC overlays to obtain adequate skid resistance (OregonDOT);
  - Requiring aggregate or top sand for TPOs, HFSTs, and PPC overlays (as applicable) be broadcast until refusal to improve initial skid resistance (NYSDOT);
  - Extending TPOs onto the bridge approaches by 10 feet such that snowplow damage to the overlay edge is inconsequential (MDOT); and
  - Seeking not only experienced contractors and crews, but also an experienced Engineer in Charge and manufacturer representative such that potential issues may be anticipated (NYSDOT).

While TPOs and HFSTs have well-understood limitations, such as their susceptibility to friction loss under high amounts of traffic and their sensitivity to poor surface preparation and ambient conditions during installation, transportation agencies are still investigating how they may be improved and how they may be better applied within bridge networks. For example, Caltrans is currently investigating if specifying a rougher surface profile will improve long-term bond strength and recently completed a study comparing the performance of a conventional PPC overlay to a system with a HFST placed on top of a PPC overlay under heavy traffic conditions. While neither system retained sufficient friction over the study, the HFST successfully protected the PPC overlay in new condition whereas the unprotected PPC overlay rutted to the original concrete, demonstrating a potential new area of application for HFSTs. As further research and case studies are completed, and specifications and practices are adjusted accordingly based on the feedback, the current challenges may be overcome and polymer overlays with improved performance and cost-effectiveness may be realized.

3.10. Summary

The general sentiment is that TPOs, HFSTs, and polymer overlays can provide a skid-resistant surface while improving resistance to moisture and chloride intrusion. The service life of TPOs and HFSTs is typically expected to be about 5 to 15 years but will vary. The thicker PPC overlays are expected to have a longer life of 15 to 25 years and provide better protection to deicer ingress, but again expected service lives vary. Service life when placed on new decks or decks in good condition is expected to be longer than when placed on decks with active reinforcement corrosion or patched corrosion-related damage.

Cracking of the new wearing surface is not common unless it is reflective of crack or joint movement in the deck. Early-age distress can be due to many different construction-related mistakes (such as poor surface preparation or improper mixing) but long-term distress is usually limited to delamination and wear. Thin polymer overlays are generally not well-suited to locations with significant studded tire or chain abrasion wear.

Skid numbers tend to be high when epoxy-based TPOs are applied but can decrease in a year or two then stabilize somewhat. However, experience is widely variable. UDOT and PennDOT have prohibited the use of flint rock in HFSTs due to their tendency to polish and poor long-term skid performance. NYSDOT noted that calcined bauxite tends to retain skid resistance well and basalt is also commonly used.

4. THREE-YEAR FIELD INVESTIGATION OF HFSTS IN MONTANA

Fourteen bridge decks located across Montana in the Billings and Missoula districts with HFSTs up to 7 years of age were inspected across a three-year period. The following presents the bridges' locations and traffic exposures, the HFST systems used, the scope of the inspections, and the observations and findings of the inspections.

4.1. Bridge Information

Fourteen bridge decks with HFSTs were inspected across a three-year period, 2020 through 2022. The characteristics of the fourteen bridges are presented in Table 8. Two of the bridges (1670 and 1682) are located within the Billings District while the remainder are located in the Missoula District. The majority of the bridges carry I-90 over various features although several of the bridges carry local roads over the Clark Fork River or I-90. The locations of the bridges are shown in the maps in Figure 4 and Figure 5.

4.1.1. Polymer Overlay Materials and Installation

For the fourteen bridges included in the field investigation, the year in which each HFST was installed and the polymer system and type of aggregate used are shown in Table 9. The two bridges in the Billings District were overlaid with HFSTs in 2015 while the bridges in the Missoula District were overlaid more recently between 2016 and 2021.

To the researchers' knowledge, all fourteen of the HFSTs investigated were constructed using Pro-Poxy Type III D.O.T. as the polymer binder. This was not confirmed for the bridges whose HFSTs were installed in 2020 or later (Bridges 14/25 and 1336) nor for Bridges 1338 and MM 49.39, but the MDT indicated that they did not plan to change materials for those projects. Pro-Poxy Type III D.O.T. is a low-modulus, low-viscosity, epoxy/urethane binder manufactured by Dayton Superior. Bridges in the Missoula District reportedly also used Pro-Poxy 45, an epoxy healer/sealer/primer also by Dayton Superior, as a primer. Reportedly, no primer was used on the Billings bridges.

The aggregates used for the HFSTs on the Billings bridges are Armorstone aggregates, which are produced by Washington Rock Quarries, Inc., and specifically from King Creek Pit. Armorstone aggregates are comprised of basalt, which is crushed and then kiln-dried, and is considered an affordable alternative to calcined bauxite aggregates, which are relatively expensive but have a high hardness that makes them well-suited for HFSTs. Based on their technical datasheet, the Armorstone aggregates have a Moh's hardness of 8 and an absorption of 0.73%. The aggregates used for the HFSTs on the Missoula bridges are typically naturally occurring calcined bauxite from the Lake Ranch quarry near the Missouri Buttes in Northeastern Wyoming. These aggregates also have a Moh's hardness of 8 and an absorption of 0.8% according to test data in the submitted material datasheets.

Prior to HFST installation, the bridges required partial- and full-depth repairs. The contractors originally used polymer repair materials due to their rapid strength gain; however, many patches reportedly failed and required re-repair prior to overlay installation. As a result, the contractors switched to cementitious repairs where possible or deemed necessary due to the size of the repair; however, a few polymer repairs remain. Cementitious deck repairs typically used HD 50, a fiber-reinforced, latex-modified, fast-setting concrete repair material by Dayton Superior, or conventional concrete from a batch truck. The polymer repair material generally used for the Missoula District bridges was Sure Patch, an epoxy repair mortar kit by Dayton Superior.

The submitted product datasheets for the polymer system, primer, and aggregates are provided in Appendix C as well as the product datasheets for the prebagged repair materials. The MDT also provided the exact dates of the HFST installations, ambient conditions during placement, and details of the partial- and full-depth repairs for select bridges included in the investigation. The installation dates and ambient conditions are summarized in Table 10, and the presence of partial- and full-depth repairs, their quantities, and the number of days between the repair work and HFST installation are provided in Table 11.

Table 8. Bridges Included in Three-Year Field Investigation

Bridge ID	District	Route Carried & Feature Crossed	Year Built or Reconstructed	ADT (vehicles per day)	Year of ADT	ADTT[1] (% of ADT)	Bridge Length (ft)
1670	Billings	I-90 WB over Greycliff Rd	1972	9522	2020	19	128
1682	Billings	I-90 WB over Bridger Creek Rd	1972	9522	2020	19	123
1459	Missoula	I-90 EB over Rt 1	2003	8044	2020	22	125
1367	Missoula	I-90 EB over railroad	2012	6415	2020	29	313
14	Missoula	Russell St NB over Clark Fork River	2019	15,747[2]	2020[2]	0[2]	463.6[2]
25	Missoula	Russell St SB over Clark Fork River	2020				
1333	Missoula	I-90 EB Ramp over St Regis River	1983	768	2020	3	260.8
1336	Missoula	I-90 WB over Clark Fork River	1982	6553	2020	27	901.9
1338	Missoula	I-90 WB over Red Hill Rd	1978	6553	2020	27	78.1
1374	Missoula	I-90 EB over Takio Lp Rd	2011	6415	2020	29	129.9
1392	Missoula	I-90 EB over Big Horn Rd	1964	9138	2020	20	124
1428	Missoula	I-90 EB over Deer Creek Rd	1998	16,309	2020	11	143
3734	Missoula	Rock Creek Rd over I-90	1972	100	2022	3	285.8
49.39 (mile marker)	Missoula	I-90 EB over Clark Fork River	2012	6415	2020	29	800.9

Notes: [1] *Average daily truck traffic (ADTT).*

[2] *Bridges 14 and 25 are the same bridge, which was built using phased construction. Traffic counts are available for the bridge, but not for the individual directions. The traffic counts for this bridge are from 2020.*

Figure 4. Map identifying the locations of the bridges selected for inspection and performance monitoring. Purple markers indicate bridges in the Billings District while blue markers indicate bridges in the Missoula District.

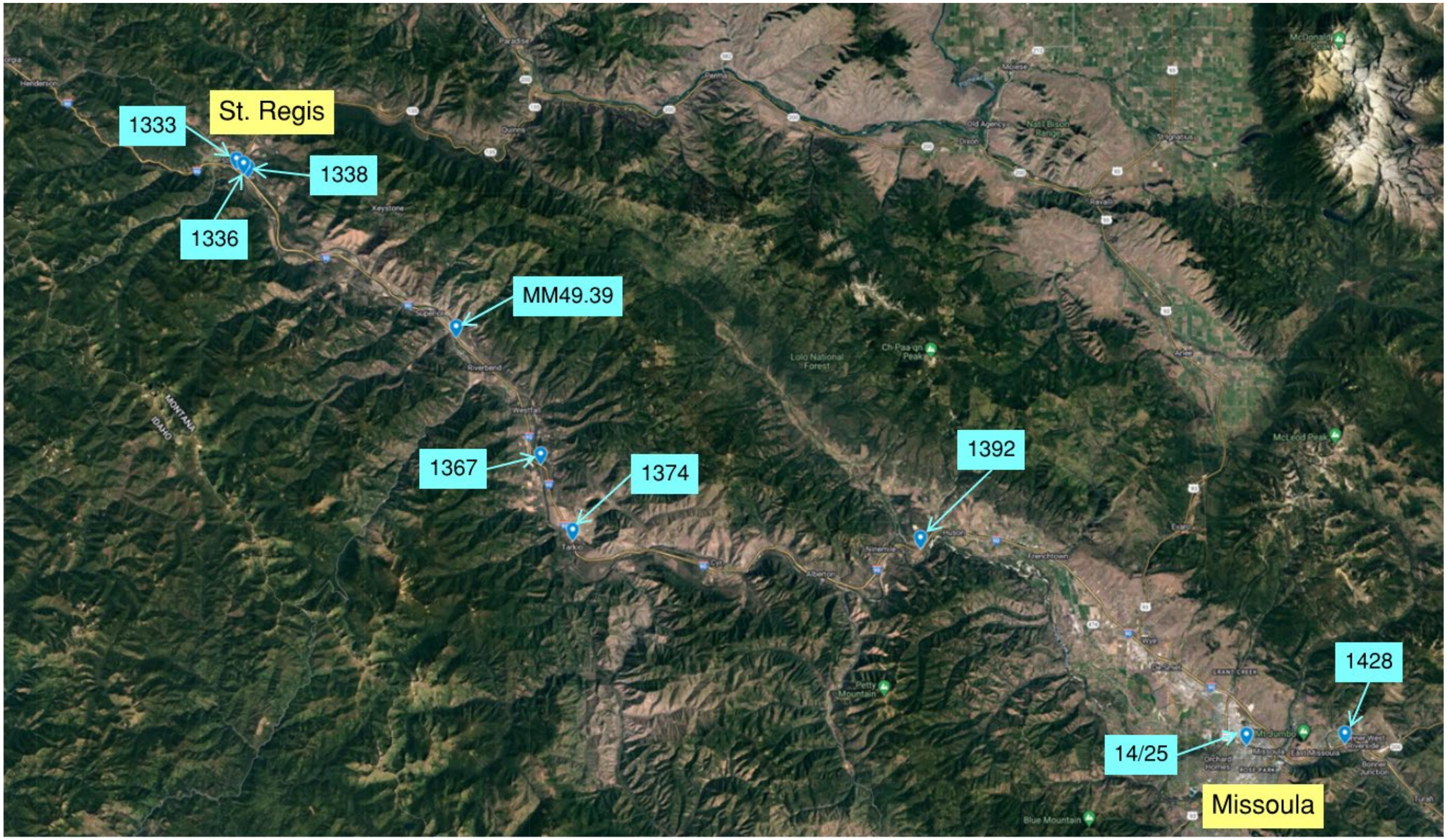

Figure 5. Close-up of the locations of bridges in the Missoula District, from Missoula, MT to St. Regis, MT.

Table 9. Summary of Available Skid Numbers for Bridge Decks with Thin Polymer Overlays[1]

Bridge ID	Year of Overlay Installation	Polymer System	Type of Aggregate	Skid Numbers							
				2014	2015	2016	2017	2018	2019	2020	2023
Bigfork	2015	Dayton Superior-Unitex	Armorstone	na	na	no test	46.5	35.8	--	--	--
Big Timber-Yellowstone River	2014	Dayton Superior-Unitex	Armorstone	83.0	60.0	no test	52.0	53.1	--	--	50.1
Kalispell	2014	Poly-Carb	Armorstone	82.0	60.0	no test	23.8	17.1	--	--	--
Roundup-Musselshell River	2014	Poly-Carb	Armorstone	81.0	60.0	no test	53.0	54.7	--	--	49.4
14	2020	--	--	na	na	na	na	na	na	--	--
25	2020	--	--	na	na	na	na	na	na	--	--
1333	2017	Pro-Poxy Type III D.O.T.	Nat. Calc. Baux. (Lake Ranch Pit)	na	na	na	--	--	--	57.7	46.9
1336	2017	--	--	na	na	na	--	--	--	--	--
1338	2021	--	--	na	na	na	na	na	na	na	--
1357	--	--	Armorstone	--	--	--	--	52.8	52.0	--	--
1359	--	--	Armorstone	--	--	--	--	54.2	55.1	--	--
1361	--	--	Armorstone	--	--	--	--	50.9	50.8	--	--
1363	--	--	Armorstone	--	--	--	--	57.7	52.8	--	--
1364	--	--	Armorstone	--	--	--	--	57.7	50.5	--	--
1367	2016	Pro-Poxy Type III D.O.T.	Nat. Calc. Baux. (Lake Ranch Pit)	na	na	--	--	--	--	55.6	50.1
1368	--	--	Armorstone	--	--	--	--	60.4	53.3	--	--
1371	--	--	Nat. Calc. Baux.	--	--	--	--	54.5	51.5	--	--
1374	2017	Pro-Poxy Type III D.O.T.	Nat. Calc. Baux.	na	na	na	--	57.0	50.6	50.4	46.8
1375	--	--	Nat. Calc. Baux.	--	--	--	--	57.0	51.5	--	--
1387	--	--	Nat. Calc. Baux.	--	--	--	--	50.6	49.7	--	--

Bridge ID	Year of Overlay Installation	Polymer System	Type of Aggregate	Skid Numbers							
				2014	2015	2016	2017	2018	2019	2020	2023
1392	2018	Pro-Poxy Type III D.O.T.	Nat. Calc. Baux. (Lake Ranch Pit)	na	na	na	na	--	--	58.1	57.1
1428	2018	Pro-Poxy Type III D.O.T.	Nat. Calc. Baux. (Lake Ranch Pit)	na	na	na	na	--	--	58.0	34.1
1459	2018	Pro-Poxy Type III D.O.T.	Nat. Calc. Baux. (Lake Ranch Pit)	na	na	na	na	--	--	58.9	50.1
1670	2015	Pro-Poxy Type III D.O.T.	Armorstone	na	--	--	--	--	--	61.2	56.1
1682	2015	Pro-Poxy Type III D.O.T.	Armorstone	na	--	--	--	--	--	56.9	55.0
3729	--	--	Nat. Calc. Baux.	--	--	--	--	45.5	58.2	--	--
3734	2018	Pro-Poxy Type III D.O.T.	Nat. Calc. Baux. (Lake Ranch Pit)	na	na	na	na	--	--	59.2	64.3
6537	--	--	Nat. Calc. Baux.	--	--	--	--	48.9	45.0	--	--
MM 49.39	--	Pro-Poxy Type III D.O.T.	Nat. Calc. Baux.	--	--	--	--	--	--	--	--

Notes: [1]*As communicated to WJE by MDT. Shaded rows indicate bridges included in the three-year field investigation conducted as part of this study. "--"indicates that the information or data is not known.*

Table 10. Ambient Conditions Recorded During Polymer Overlay Placement[1]

Bridge ID	Dates of Installation	Hi Temp. (°F)	Low Temp. (°F)	RH (%)	Wind (mph)	Other Notes
Detailed Investigation with Laboratory Testing						
1459	7/31/2018	85	53	38	4	Most of the wind was between 2 pm and 9 pm
	8/1/2018	95	55	34	4	Most of the wind was between 1 pm and 9 pm
	8/2/2018	92	52	27	7	Most of the wind was between 2 pm and 9 pm avg 12 mph
	8/3/2018	83	49	29	11	Most of the wind was between 2pm and 9pm avg 19 mph
Visual Inspection Only						
1333	9/26/2017	65	45	68	2	
	9/27/2017	72	39	60	1	
	9/28/2017	74	37	59	1	
1392	6/12/2018	71	36	39	3	Most of the wind was between 2pm and 9pm avg 5 mph
	6/13/2018	80	49	39	7	Most of the wind was between 2 pm and 9 pm avg 11 mph
1428	7/9/2018	93	51	39	5	Most of the wind was between 2pm and 9pm avg 9 mph
	7/10/2018	84	56	35	11	
3734	7/24/2018	93	50	24	6	Most of the wind was between 2pm and 9pm avg 10 mph
	7/25/2018	91	54	29	6	

Notes: [1]*Information is presented as communicated to WJE by MDT.*

Table 11. Description of Partial- and Full-Depth Repairs at Inspected Bridge Decks[1]

Bridge ID	Partial-Depth Repairs:			Full-Depth Repairs:		
	Present?	Quantity	Min. Time Btwn. Repair and Installation	Present?	Quantity	Min. Time Btwn. Repair and Installation
1670	yes	--	--	no	NA	NA
1682	no	NA	NA	yes	--	--
1459	yes	--	--	no	NA	NA
1367	yes	--	--	yes	61 sq yd	--
14	--	--	--	--	--	--
25	--	--	--	--	--	--
1333	yes	42.3 sq yd	4 days	no	NA	NA
1336	--	--	--	--	--	--
1338	--	--	--	--	--	--
1374	yes	--	--	no	NA	NA
1392	yes	20.9 sq yd	54 days	no	NA	NA
1428	yes	0.2 sq yd	75 days	no	NA	NA
3734	yes	65.7 sq yd	70 days	yes	13.9 sq yd	70 days
MM 49.39	--	--	--	--	--	--

Notes: [1]*Information is as communicated to WJE by MDT. "--" indicates that the information is not known.*

4.1.2. HFST Distress Observed by MDT

Nine of the fourteen bridges in the investigation were identified by the MDT as priorities for investigation based on the conditions and ages of their HFSTs. MDT personnel had previously inspected the HFSTs on select decks for cracking, spalling, overlay debonding, and wear, and a summary of their observations is provided in Table 12.

None of the HFSTs showed signs of debonding, but both of the Billings bridges demonstrated spalling and widespread reflective cracking in the HFST above repairs. Failure of polymer patches and reflective cracking were identified as widespread, key issues by MDT personnel, and due to these issues, they reportedly switched to cementitious materials for patching. The HFSTs of the bridges in the Missoula District additionally showed some transverse reflective cracking, wear in the wheel paths of the treatment, and aggregate pop-outs. While the HFSTs are thick enough that surface aggregate pop-outs did not expose the deck, 1/8- to 1/4-inch pockmarks were left behind. Aggregate pop-outs were particularly prevalent in the wheel paths and MDT personnel communicated that they suspected pop-outs primarily occur due to the use of studded tires. Photographs showing examples of transverse cracking, aggregate pop-out and overlay wear, and reflective cracking due to an underlying partial-depth repair are shown in Figure 6 through Figure 8.

Table 12. Notes on HFST Conditions Based on 2020 Inspections by MDT Personnel

Bridge ID	Spalling	Reflective Cracking of HFST over Repairs	Reflective Transverse Cracking in HFST	Severity of Wear in Wheel Paths
1670	Present	Severe	--	--
1682	Present	Severe	--	--
1459	None	None	None	Driving Lane: Moderate, with pop-outs Passing Lane: Light, no pop-outs
1367	None	None	Moderate	Driving Lane: Moderate, with pop-outs Passing Lane: Light, no pop-outs
1333	None	None	None	Driving Lane: Moderate, with pop-outs Passing Lane: Light, no pop-outs
1374	None	None	Moderate	Driving Lane: Moderate, with pop-outs Passing Lane: Light, no pop-outs
1392	None	None	None	No wear
1428	None	None	Light; present only in driving lane	Driving Lane: Light, with pop-outs Passing Lane: Light, no pop-outs
3734	None	None	None	No wear

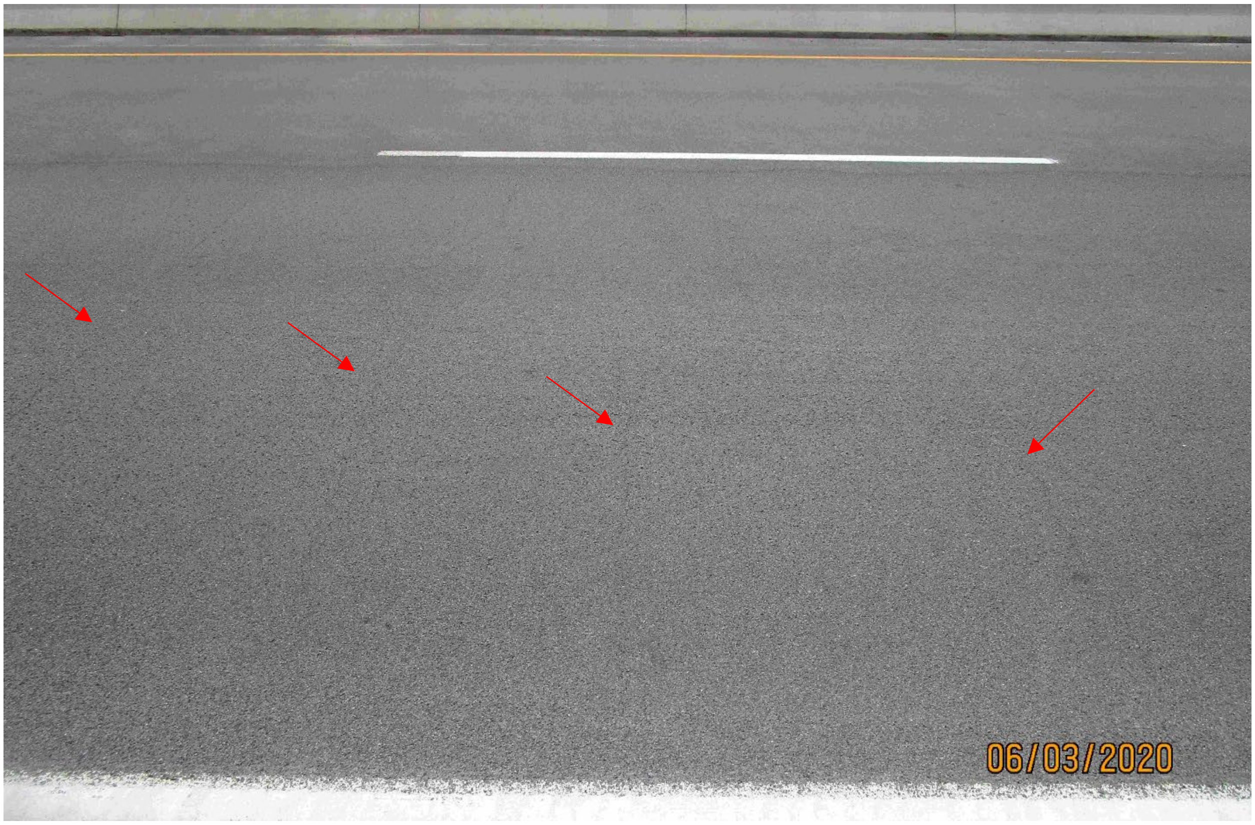

Figure 6. Transverse cracks in a HFST applied to MDT Bridge 1374, which is in the Missoula District. Image provided by MDT. Red arrows added by WJE to point out the transverse cracks.

Figure 7. A transverse crack and aggregate pop-out in the HFST applied to MDT Bridge 1374, which is in the Missoula District. Image provided by MDT.

Figure 8. Reflective cracking and spalling over partial-depth repairs on Bridge 1682 in the Billings District. Image provided by MDT.

4.1.3. Skid Resistance of HFSTs

The MDT tracked the skid resistance of a number of bridge decks that had been given HFSTs until 2021/2022, at which time the MDT's in-house equipment could no longer be used. Skid resistance testing was conducted following ASTM E274, *Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire*. The skid resistance data available for the bridges investigated in this study is summarized in Table 9, which shows the average skid number measured for each bridge. Table 9 also includes the skid resistance data and HFST information for a number of bridges that were not included in this study, including the four bridges that caused the MDT to initiate this study (Bigfork; Big Timber-Yellowstone River; Kalispell; and Roundup-Musselshell River). The skid numbers for these additional bridges are presented as communicated to WJE as supplemental information.

The skid resistance data for the bridges included in this study is limited. Skid resistance data from 2020 is available for most of the bridges investigated, but only Bridge 1374 has skid resistance data from previous years (2018 and 2019). For the 2020 data, skid numbers were provided for both the driving and the passing lanes on each bridge, and this more detailed data is shown in Table 13. The skid numbers of the driving and passing lanes differed by as little as 3.6 to as much as 12.7. The median difference in 2020 was 9.8 and the driving lane always had a lower skid number than the passing lane.

International Cybernetics was contracted to complete skid resistance testing on September 12 to 14, 2023 and the test report is included as Appendix H. Weather conditions were clear to partly cloudy with ambient temperatures ranging from 53°F to 93°F. Testing was performed using an ICC SFT 5041 Pavement Friction Tester in accordance with ASTM E274 and AASHTO T-242 using a ribbed tire (AASHTO M261) in the left wheel path of each lane. The skid resistance data per lane from 2023 testing is shown in Table 14. The skid numbers of the driving and passing lanes differed by as little as 3.0 to as much as 19.3. The median difference in 2023 was 14.2 and the driving lane always had a lower skid number than the passing lane. On average, the skid numbers in 2023 were 13.8 % lower than the corresponding value in 2020. The driving lane of Bridge 1428 had a skid number of 24.4 which is below the commonly required minimum of 30 to 35. Five test cycles were performed and averaged for each bridge with a test interval of 528 feet and target or normalized speed of 40 mph.

Table 13. 2020 Skid Numbers for Individual Bridge Lanes[1]

Bridge ID	HFST Age in 2020	Driving Lane	Passing Lane
1670	5 years	54.8	67.5
1682	5 years	54.4	59.4
1459	2 years	54.0	63.8
1367	4 years	53.8	57.4
1333[2]	3 years	54.6 (off-ramp)	60.8 (on-ramp)
1374	3 years	45.5	55.2
1392	2 years	53.0	63.2
1428	2 years	58.0	Closed
3734[3]	2 years	58.4 (NB)	60.0 (SB)

Notes: [1] *Information is as communicated to WJE by MDT.*

[2] *Bridge 1333 is a two-way ramp leading onto I-90. The skid numbers under the driving and passing lanes are the off-ramp and on-ramp lanes, respectively.*

[3]Bridge 3734 is a north/south overpass over I-90. The skid numbers under the driving and passing lanes are the NB and SB lanes, respectively.

Table 14. 2023 Skid Numbers for Individual Bridge Lanes

Bridge ID	HFST Age in 2023	Driving Lane	Passing Lane
1670	8 years	46.8	65.3
1682	8 years	51.3	58.7
1459	5 years	43.0	57.2
1367	7 years	43.4	56.8
1333[1]	6 years	Not tested (off-ramp)	46.9 (on-ramp)
1374	7 years	38.7	54.8
1392	5 years	55.6	58.6
1428	5 years	24.4	43.7
3734[2]	5 years	62.4 (NB)	66.2 (SB)
1137 NB[3]	unknown	40.7	46.8
1137 SB[3]	unknown	44.1	53.7
1138 NB[3]	~ 2 years	38.4	42.8
1138 SB[3]	~ 2 years	34.6	50.8
1139 NB[3]	~ 1 year	34.4	52.8
1139 SB[3]	~ 1 year	38.6	42.7

Source: [1]Bridge 1333 is a two-way ramp leading onto I-90. The skid numbers under the driving and passing lanes are the off-ramp and on-ramp lanes, respectively.

[2]Bridge 3734 is a north/south overpass over I-90. The skid numbers under the driving and passing lanes are the NB and SB lanes, respectively.

[3]Bridges 1137, 1138, and 1139 were added to the scope of the 2023 skid testing, but are otherwise outside the scope of this project. Records on InfoBridge indicate that Bridge 1137 has a monolithic concrete wearing surface while Bridges 1138 and 1139 have epoxy overlays as of the inspections in 2021 and 2022. All three bridges had an ADT of 2679 and ADTT of 19% as of 2021.

Figure 10 provides the skid number versus time for three of the original four HFSTs placed by MDT. The skid numbers for the Bigfork Bridge are not shown since the initial skid resistance of its HFST was not recorded and this HFST was not selected for skid resistance testing in 2023 due to its low average skid number of 35.8 in 2018. As a point for comparison, skid numbers ranging from the low to high 40s with averages in the mid-40s have been measured for bare concrete bridge decks[1].

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[1] Personal communication with International Cybernetics.

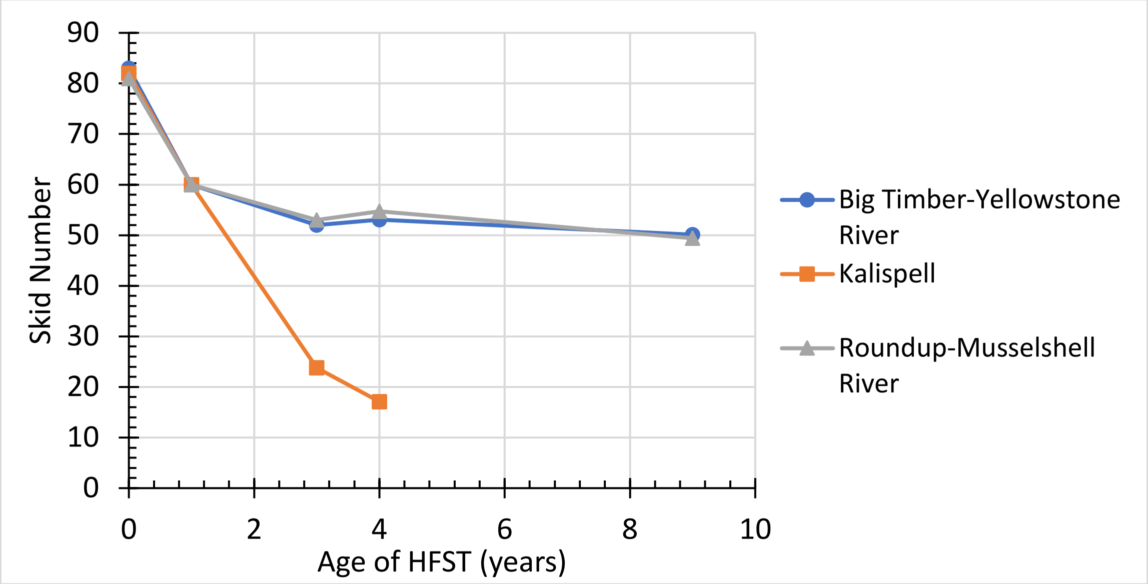


Figure 9. Plot of average skid numbers against HFST age for Big Timber (Dayton Superior-Unitex) and Kalispell and Roundup (Poly-carb), all of which used Armorstone aggregate.

4.2. Bridge Inspections

Bridge inspections were conducted August 24 to 28, 2020; August 2 to 3, 2021; and September 12 to 15, 2022. Four bridges (Bridges 1670, 1682, 1459, and 1367) were selected for detailed investigations based on their locations and the conditions of their HFSTs. Traffic control was provided for these bridges and the detailed investigations consisted of a visual inspection, delamination survey, and coring for laboratory testing.

The remaining bridges underwent visual inspections. If lane closures were present, or if traffic volumes and speeds were low enough that a survey could be performed safely, then a visual inspection of the HFST and deck soffit and a chain-drag were performed. In some instances, only the deck soffit could be inspected. Conditions were recorded using photographs and WJE's in-house software Plannotate. The distress maps drawn in Plannotate are provided in Appendix D.

Twelve bridges were inspected in 2020. In 2021, a preliminary visual inspection of the HFSTs on the Billings bridges was conducted and because minimal to no deterioration progression was observed between 2020 and 2021, additional laboratory testing of the cores collected in 2020 was performed instead of the remaining 2021 bridge inspections. In 2022, the final year of the study, thirteen of the bridges were inspected; the final bridge (MM 49.39) was inaccessible without traffic control. The bridge inspection schedule is summarized in Table 15.

Table 15. Bridge Inspections Conducted 2020 to 2022[1]

Bridge ID	Aug. 24-28, 2020	Aug. 2-3, 2021	Sep. 12-15, 2022
1670	Detailed	Visual	Detailed
1682	Detailed	Visual	Detailed
1459	Detailed	--	Detailed
1367	Detailed	--	Detailed

Bridge ID	Aug. 24-28, 2020	Aug. 2-3, 2021	Sep. 12-15, 2022
14	Visual	--	Visual
25	Visual	--	Visual
1333	Visual	--	Visual
1336	--	--	Visual
1338	--	--	Visual
1374	Visual	--	Visual
1392	Visual	--	Visual
1428	Visual	--	Visual
3734	Visual	--	Visual
MM 49.39	Visual	--	--

Notes: [1] *"--" indicates no inspection was performed.*

4.3. Bridges Undergoing Detailed Inspections

The following presents the conditions found at each of the four bridges selected for detailed investigations.

4.3.1. Bridge 1670 (Billings)

Overview photographs of the HFST on Bridge 1670 from 2020 and 2022 are shown in Figure 10 and Figure 11, respectively.

Figure 10. Overview of the driving lane and shoulder for Bridge 1670 in 2020.

Figure 11. Overview of the driving lane and shoulder for Bridge 1670 in 2022.

4.3.1.1. 2020 Inspection

In 2020, the HFST showed the following distress at an age of 5 years:

- Wear and aggregate pop-out, particularly in the driving lane wheel paths (Figure 12) and less wear in the passing lane and shoulder (Figure 13);
- Reflective cracking at patch repair boundaries (Figure 14);
- Limited spalling of the overlay, near the patch boundaries (Figure 14) or in the form of small "popout spalls;"
- Loss of the HFST topping at the edge at the bridge approach joint due to snow plow impact (Figure 15); and
- Delamination at the reinforcing steel due to continued corrosion of embedded deck reinforcement (Figure 15).

The deck soffit had small amounts of efflorescence but generally did not show distress (Figure 16). However, the abutments exhibited significant moisture intrusion and efflorescence with pattern (horizontal and vertical) cracking (Figure 17). This distress is characteristic of alkali-silica reaction, but petrography would be required to confirm the distress mechanism at work in the abutments (outside scope of this work).

Figure 12. Photograph of wear and aggregate pop-out in driving lane of Bridge 1670 in 2020.

Figure 13. Photograph of minimal wear in the shoulder of Bridge 1670 compared to the driving lane in 2020.

Figure 14. Reflective cracking in the driving lane of the HFST on Bridge 1670 and several spalls of the HFST, as of 2020.

Figure 15. Spall exposing rebar and delamination on Bridge 1670, as seen in 2020 inspection. Loss of HFST at the bridge edge due to snow plow damage is visible in the upper left.

Figure 16. Image showing the typical conditions of the soffit of Bridge 1670 in 2020. Small amounts of efflorescence were present.

Figure 17. Photograph showing moisture efflorescence, cracking, and distress on the abutments of Bridge 1670 as of 2020.

4.3.1.2. 2022 Inspection

In 2022, the HFST showed the following distress and progression in deterioration at an age of 8 years:

- Wear and aggregate pop-out remained prevalent in the wheel paths compared to the lanes or shoulder with less traffic exposure. Microscopic examination showed that aggregate particles were often fractured to the approximate level of the binder resin but remained well-bonded. Some microscopic cracking of the binder resin was noted, and some pock-mark type holes were seen where particles that had not been fully adhered had dislodged as small agglomerations. Microscopic examination of the passing lane showed much less fracturing of the aggregate and less aggregate loss. The shoulder area looked mostly untouched with the aggregate particles mostly intact and well-adhered.
- While several patches remained sound, most patches were delaminated around their perimeters where reflective cracking had been identified in 2020, as shown by the field notes in Appendix D and in Figure 18.
- The number of small, pop-out spalls in the HFST increased greatly between 2020 and 2022. The pop-out spalls were primarily in the driving lane and at the bridge approach at the shoulder.

- The approach edge at the bridge entrance still exhibited damage and loss of the HFST topping at local areas due to snow plow impact (Figure 19). The leading edge of the bridge was often damaged as well as local gouge areas 6 to 10 feet from the bridge joint where the plow blade had bounced.
- One transverse crack across the travel lanes of the bridge was observed above the location of one of the bents (possibly an active moment related crack).

The underside of the deck remained in good condition with no significant leakage through the deck cracks. The pattern cracking and heavy efflorescence in the abutments was still present.

Figure 18. Photograph from 2022 inspection of Bridge 1670 showing delaminations around repair patches' edges that had reflective cracking previously. The centers of the patches remained sound.

Figure 19. Photograph of snow plow damage and gouging at the entrance joint onto Bridge 1670 in the 2022 inspection.

4.3.2. Bridge 1682 (Billings)

Overview photographs of the HFST on Bridge 1682 from 2020 and 2022 are shown in Figure 20 and Figure 21, respectively.

Figure 20. Overview of Bridge 1682 in 2020. Note photographer is facing in the direction of traffic.

Figure 21. Overview of Bridge 1682 in 2022. Note photographer is facing into oncoming traffic.

4.3.2.1. 2020 Inspection

The conditions observed at Bridge 1682 in 2020 when the HFST was at an age of 5 years were very similar to those observed at Bridge 1670 in 2020. The HFST demonstrated aggregate pop-out, reflective cracking over deck repairs with associated delaminations and spalls (Figure 22), and some spalling or damage at the edge of the deck (Figure 23). One area in particular had severe aggregate pop-out (Figure 24). The deck soffit had minimal distress with some small amounts of efflorescence. The abutments and girder ends showed distress characterized by white deposits and loss of paste (Figure 25).

Figure 22. Photograph of reflective cracking and spalling of deck repairs and HFST on Bridge 1682 in 2020.

Figure 23. Abrasion/impact damage of the HFST at the leading approach edge of the HFST on Bridge 1682 in 2020.


Figure 24. Area of overlay raveling and pockmarking in the HFST at Bridge 1682 in 2020.

Figure 25. Close-up of distress on abutment and girder end at Bridge 1682 in 2020 suggesting significant moisture and deicer exposure.

4.3.2.2. 2022 Inspection

In 2022, the HFST showed the following distress and progression in deterioration at an age of 8 years:

- In 2020, the delaminations present were typically very small and located at the corners of the patch repairs. In 2022, the delaminations progressed and were generally present along the reflective cracking around the entire perimeter of the patches. The number of spalls at the patch edges also increased between 2020 and 2022. Figure 26 shows a photograph of the delaminated areas and spalls around patched areas. Further, new delaminated areas on the order of 1 or 2 square feet in size were also found in the shoulders where previous patches were not present.
- One transverse crack was also found approximately near one of the bents and extended through a patched and delaminated area in the driving lane (Figure 27).
- As seen in 2020, the leading edge of the bridge had an area where the HFST had been completely worn away (Figure 28), likely due to damage from snowplows and other vehicles. The extent of the worn area did not increase. The opposite joint where traffic exits the bridge also demonstrated some loss of the HFST.

Unlike Bridge 1670, the HFST on Bridge 1682 did not contain multiple small spalls. The underside of Bridge 1682 was generally in good condition although one area had a short crack with moderate efflorescence (Figure 29). The abutments and girder ends showed the same distress (i.e., efflorescence and loss of paste) as seen in 2020.

Figure 26. Reflective cracking, delaminations, and spalls around patch repairs in driving lane of Bridge 1682 in 2022.

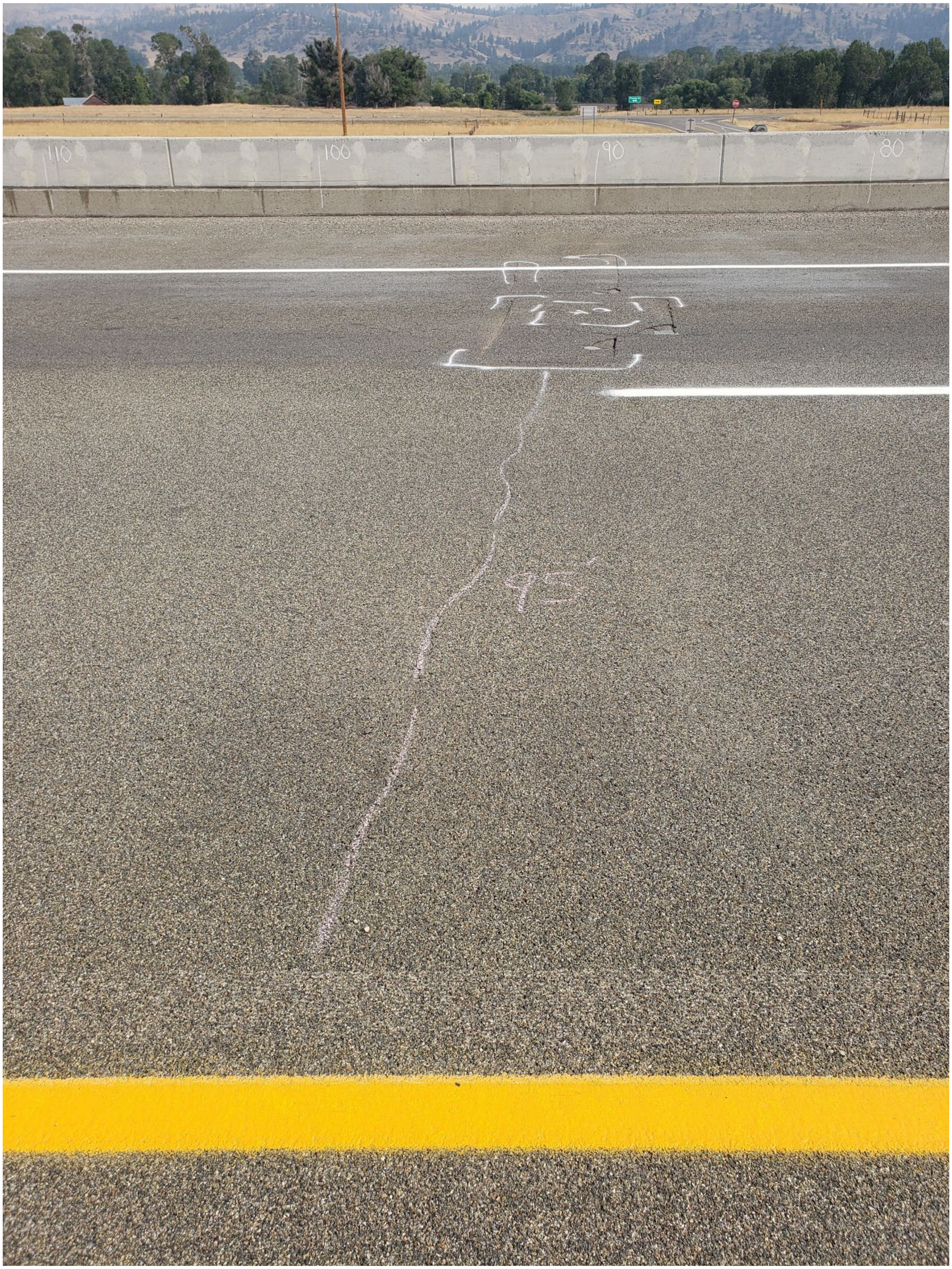


Figure 27. Transverse crack near bent in Bridge 1682 in 2022.

Figure 28. Area on leading edge of Bridge 1682 in 2022 where HFST was fully worn away.

Figure 29. Area with a short, efflorescing crack on the soffit of Bridge 1682 in 2022.

4.3.3. Bridge 1459 (Missoula)

Overview photographs of the HFST on Bridge 1459 from 2020 and 2022 are shown in Figure 30 and Figure 31, respectively.

Figure 30. Overview of HFST on Bridge 1459 in 2020. The lap in the overlay is identified by the red arrow.

Figure 31. Overview of HFST on Bridge 1459 in 2022 (staining due to recent coring).

4.3.3.1. 2020 Inspection

In 2020, the HFST showed the following distress and features at an age of 2 years:

- Aggregate pop-out (Figure 32);
- Cracking at the bridge end (Figure 33);
- Small gouged areas in the overlay (Figure 34); and
- An overlay lap with a clear difference in overlay wear at the boundary (Figure 35).

No reflective cracking in the HFST was observed and the HFST on Bridge 1459 showed relatively little distress compared to the HFSTs on the bridges in the Billings District in 2020. The soffit showed regular transverse cracks with efflorescence (Figure 36).

Figure 32. Photograph showing aggregate loss in the driving lane of Bridge 1459 in 2020.

Figure 33. Cracking and spalling of the HFST at the end of Bridge 1459 in 2020.

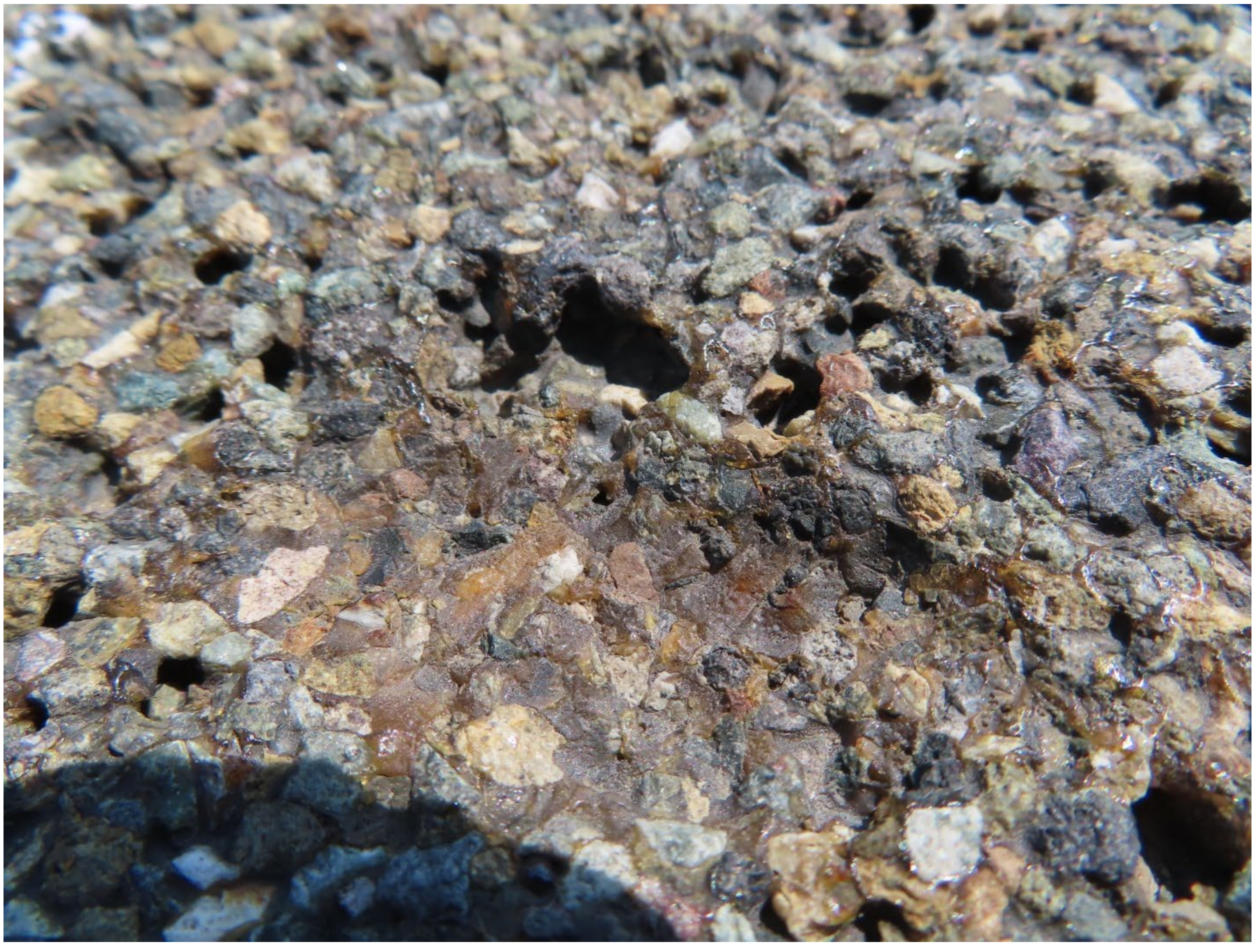


Figure 34. Close-up of the gouged area (likely plow damage) in the HFST of Bridge 1459 in 2020.

Figure 35. Close-up of the construction lap in the overlay of Bridge 1459 showing a clear difference in wear and texture in 2020.


Figure 36. Photograph showing transverse cracking and efflorescence on the deck soffit of Bridge 1459 in 2020.

4.3.3.2. 2022 Inspection

In 2022, the HFST showed the following distress and progression in deterioration at an age of 5 years:

- The wheel paths of the driving lane had an open texture due to traffic wear. The passing lane and shoulder areas showed much less wear than the driving lane. Microscopic examination showed that the tops of some aggregate particles were worn but the aggregates appeared to be well-bonded. The open texture was caused by particle-depth holes in the surface due to loss of surface aggregate or small agglomerations of aggregate. In some instances, aggregate loss exposed entrapped voids where the overlay was not fully consolidated. The aggregate loss and pock marks left behind appeared to be a result of incomplete consolidation rather than loss and dislodging of well-consolidated overlay material. Close microscopic examination of the shoulder area showed that entrapped consolidation-type voids were present below the top surface. Therefore, traffic wear likely dislodged some surface aggregate and exposed areas of incomplete consolidation. A photograph of the surface in one of the wheel paths in the driving lane is shown in Figure 37.
- The edges of the HFST at the bridge ends, which had been cracked in 2020, showed signs of additional delamination and more spalling in 2022 (Figure 38).
- One longitudinal crack was present in the passing lane at the east end of the bridge. The crack was approximately 10 feet long.

The numerous transverse cracks with efflorescence on the soffit of the deck generally appeared unchanged from 2020 and reflective cracking in the HFST was not visible on the top surface. Overall, the HFST of Bridge 1459 did not appear to have experienced much progressive deterioration between 2020 and 2022.

Figure 37. Close-up of the wearing surface in one of the wheel paths of the driving lane of Bridge 1459 in 2022 at prior core hole location. Polymer concrete is holding up well.

Figure 38. Cracking, spalling, and delamination of the HFST along the end of Bridge 1459 in 2022. Note this is the same joint as that shown in Figure 33 from 2020.

4.3.4. Bridge 1367 (Missoula)

Overview photographs of the HFST on Bridge 1367 from 2020 and 2022 are shown in Figure 39 and Figure 40, respectively.

Figure 39. Overview of HFST on Bridge 1367 in 2020. Note photographer is facing into oncoming traffic.

Figure 40. Overview of HFST approach on Bridge 1367 in 2022. Note photographer is facing in the direction of traffic.

4.3.4.1. 2020 Inspection

In 2020, the HFST showed the following types of distress at an age of 4 years:

- Widespread overlay aggregate loss and surface polishing of the HFST (Figure 41), especially in the driving lane (Figure 42) compared to the shoulder (Figure 43);
- Minor spalling of the HFST (Figure 44);
- Minor wear and spalling of the HFST at its edges at the bridge ends; and
- Transverse cracking (Figure 44), particularly near full-depth and partial-depth repairs.

Additionally, the soffit had regular transverse cracking with efflorescence and multiple full-depth repairs of large areas, both of which are shown in Figure 45.

Figure 41. Photograph showing aggregate loss and polishing of the HFST in the driving lane of Bridge 1367 in 2020.

Figure 42. Close-up of overlay pop-out and wear in the driving lane of Bridge 1367 in 2020.

Figure 43. Close-up showing surface roughness and surface aggregates in the shoulder of Bridge 1367 in 2020.

Figure 44. Small spall and transverse cracks in HFST on Bridge 1367 in 2020.

Figure 45. Photograph of soffit of Bridge 1367 in 2020 showing transverse cracks with efflorescence and full-depth repairs.

4.3.4.2. 2022 Inspection

In 2022, the HFST showed the following distress and progression in deterioration at an age of 7 years:

- The wheel paths in the driving lane had an open texture due to traffic wear. The wear appeared to be similar in nature to that observed at Bridge 1459 in the 2022 inspection. Close-ups of the HFST surface showing the open texture in the wheel path versus the rough texture of the shoulder are shown in Figure 46 and Figure 47, respectively. Angled close-ups of the HFST in between the wheel paths (Figure 48), in the driving lane wheel path (Figure 49), and in the shoulder (Figure 50) are also provided to show the difference in surface roughness and condition.
- The inspectors observed more transverse cracks (Figure 51) in the HFST in 2022 than in 2020; however lighting conditions affect ease of identification of cracking such that cracking may have been early age and not recent. Despite the relatively large number of transverse cracks compared to the other bridges inspected, no overlay delaminations along cracks were found.
- The small spalls in the HFST observed in 2020 did not appear to have grown; only two small spalls, one of which is shown in Figure 52, were observed in 2022.

- Minor wear and spalling was still present at the edges of the HFSTs at the bridge ends.

The underside of the deck was not inspected in 2022 since the bridge crosses over a railroad with limited access.

Figure 46. Close-up of wheel path surface on Bridge 1367 in 2022.

Figure 47. Close-up of shoulder surface on Bridge 1367 in 2022.

Figure 48. Close-up of HFST surface between wheel paths in driving lane on Bridge 1367 in 2022.

Figure 49. Close-up of HFST surface in right wheel path of driving lane on Bridge 1367 in 2022. Note polished surface and pockets of local loss of overlay.

Figure 50. Close-up of HFST surface in shoulder on Bridge 1367 in 2022. Note aggregate remains proud and intact.

Figure 51. Photograph showing transverse cracking in the driving lane of Bridge 1367 in 2022. Some transverse cracks are identified with red arrows.

Figure 52. Close-up of small spall (gouge) in HFST of Bridge 1367 in 2022.

4.4. Bridges Undergoing Visual Inspections

4.4.1. 2020 Inspections

The HFSTs on the bridges that underwent visual inspections in 2020 varied in condition. The recently completed HFSTs on Bridges 14 and 25 (less than 1 year old) were in the best condition, with no reflective cracking observed. Some cracks in the soffits of these bridges showed evidence of moisture leakage and efflorescence while others were dry. Additionally, these were the only two bridges where overlay aggregate loss or pop-out and wear were not yet observed.

In contrast, the 3-year-old HFST on Bridge 1374 was in relatively poor condition and showed regular transverse cracking with a spacing of 4 feet on center as well as the overlay aggregate loss and wear in the wheel paths that was similar to the conditions observed at the bridges undergoing detailed investigation.

The HFSTs on the remaining bridges were generally in good condition. All of them showed signs of overlay aggregate loss in the wheel paths, although the 2-year-old HFSTs on Bridges 1392 and 3734 had relatively minor amounts of wear. Bridges 1333, 1428, and MM49.39 had several transverse cracks, but these cracks were in localized regions instead of spaced regularly along the deck. Bridge 3734 had a small spall much like the spall observed on Bridge 1367 and shown in Figure 44. None of the bridges showed reflective cracking at repairs. The conditions observed at each bridge deck are summarized in Table 16.

Table 16. Summary of Visual Observations of HFSTs on Decks Subjected to Visual Inspection in 2020

Bridge ID	HFST Age	Wear in Wheel Paths	Transverse Cracking in HFST	Spalling of HFST	Reflective Cracking at Patch Repairs
14[1]	< 1 yr	--	--	--	--
25[1]	< 1 yr	--	--	--	--
1333	3 yrs	Typical wear	Infrequent cracks	--	--
1336[2]	3 yrs	NA	NA	NA	NA
1338[2]	NA	NA	NA	NA	NA
1374[1]	3 yrs	Typical wear	Regularly spaced cracks	--	--
1392[1]	2 yrs	Minor wear	--	--	--
1428[1]	2 yrs	Typical wear	Cracks over bents	--	--
3734[1]	2 yrs	Minor wear	--	--	Bump at one patch boundary but no cracking
MM49.39	unknown	Typical wear	Infrequent cracks	Small spalled area	--

Notes: [1] *Indicates traffic control was unavailable and so a limited visual inspection was conducted.*

[2] *Bridge not inspected in 2020.*

The HFSTs commonly encroached on top of the joint armor, if armor was present, and aggregates from the overlay could be found as debris in the seal, as observed on Bridge 25 and shown in Figure 53. Also, large patches of a bituminous material were present at the entrances to Bridges 1333 and 1374, as shown in Figure 54.

Figure 53. Photograph showing aggregate debris in an armored joint on Bridge 25 and encroachment of the polymer overlay onto the joint armor.

Figure 54. Large bituminous patch present at the entrance onto Bridge 1374.

4.4.2. 2022 Inspections

The distress in the HFSTs observed in 2022 is summarized in Table 17. Of the HFSTs inspected in 2022, the HFST on Bridge 1338 was in the best condition with no visible cracking, wear, or spalls and no delaminations in the surveyed portion. This is expected since the HFST on Bridge 1338 was installed in 2021, just one year prior.

The HFST on Bridge 3734 remained in relatively good condition compared to the other HFSTs. Although delaminations were detected, no cracks or spalls in the HFST were present. When the HFST was chipped away from several areas identified as delaminated or unsound, it was found that the HFST was still well-bonded to the surface (Figure 55); the delaminations therefore appeared to be at the level of the reinforcing steel and due to continued corrosion rather than overlay disbondment.

The HFST on Bridge 1392 also remained in relatively good condition. The wear did not appear to progress and remained minor, and the open texture observed on other HFSTs was not present. Transverse cracks were noted in 2022, which had not been observed in 2020, and were likely located over or near the bents.

The HFSTs on Bridges 14 and 25, which had been installed in 2020 and were the HFSTs in the best condition in 2020, had developed regularly spaced transverse cracks with an average spacing of approximately 5 to 10 feet. The HFSTs had also developed the open texture and wear that was typically observed in other older HFSTs. Transverse cracking and wear in the SB lanes (Bridge 25) are shown in Figure 56.

The condition of the HFST on Bridge 1374, which was in relatively poor condition compared to the other HFSTs observed in 2020, did not appear to have changed in 2022. Figure 57 and Figure 58 show one section of Bridge 1374 in 2020 and 2022 respectively for reference. Regular transverse cracks were present in both years and at a close spacing of less than 5 feet and even as low as 1 or 2 feet in some locations (Figure 58). Wear and open texture in the wheel paths of the driving lane are also apparent in the photographs. The HFST on Bridge 1374 remained in relatively poor condition compared to the other HFSTs in 2022; however, the polymer topping was still well bonded to the deck and generally intact.

The HFST on Bridge 1428 (installed in 2018) appeared to be in a similar condition to Bridge 1374 (installed in 2017), with a similar amount of wear. Regularly-spaced transverse cracks, which had not been observed in 2020, were observed in 2022. They may have developed between 2020 and 2022, or been easier to see because of the relatively wet weather during the 2022 inspection or because the cracks increased in width.

The HFSTs on Bridges 1333 (installed in 2017) and 1336 (also installed in 2017) both had cracking and unsound areas, likely caused by continued corrosion of reinforcement rather than overlay disbondment, as was observed at Bridge 3734 (installed in 2018). The unsound areas on Bridge 1333 tended to occur along or at the ends of cracks (Figure 59), and a greater number of cracks were observed in 2022 than in 2020. While most of the cracks were transverse, some were diagonal and aligned with the skew of the bridge. Cracking of the HFST on Bridge 1336 was relatively infrequent compared to other bridges with a spacing of approximately 10 or more feet. The delamination survey on Bridge 1336 was only conducted along the left wheel path of the passing lane and multiple small delaminations were found. Like those on Bridge 1333, the delaminations on Bridge 1336 also typically aligned with cracks in the deck and likely represent areas of reinforcing steel corrosion.

Table 17. Summary of Visual Observations of HFSTs on Decks Subjected to Visual Inspection in 2022

Bridge ID	HFST Age	Wear in Wheel Paths	Transverse Cracking in HFST	Delaminations & Spalling of HFST	Reflective Cracking at Patch Repairs
14[1]	2 yrs	Typical wear[2]	Regularly-spaced cracks[2]	--	--
25[1]	2 yrs	Typical wear	Regularly-spaced cracks, avg. spacing about 5 to 10 feet	--	--
1333	5 yrs	Typical wear	Irregularly-spaced cracks	Multiple delams present	--
1336	5 yrs	Minor wear	Regularly-spaced cracks, avg. spacing about 10 or more feet	Multiple small delams present	--
1338	1 yr	--	--	--	--
1374[1]	5 yrs	Typical wear	Regularly-spaced cracks, avg. spacing less than 5 feet	--	--
1392[1]	4 yrs	Minor wear	Irregularly-spaced cracks	--	--
1428[1]	4 yrs	Typical wear	Regularly-spaced cracks	--	--
3734	4 yrs	Minor wear	--	Multiple small delams (~1 to 2 ft^2) present	--
MM49.39	unknown	NA	NA	NA	NA

Notes: [1] *Indicates inspection was limited to visual inspection.*

[2] *Only the SB lanes were inspected in detail; conditions of Bridge 14 on the other side of the median are expected to be similar based on the cursory visual inspection of the NB lanes.*

Figure 55. Area identified as delaminated on Bridge 3734 in 2022. The “spall” at the center is due to chipping the HFST away to determine if the delamination was at the HFST-deck interface or due to reinforcement corrosion.

Figure 56. Photograph showing transverse cracking and wear of the SB lanes (foreground) on Bridge 25 in 2022. The NB lanes (Bridge 14) are in the background. Transverse cracks are identified with red arrows.

Figure 57. Photograph of a location on Bridge 1374 showing transverse cracking in 2020.

Figure 58. Photograph from 2022 of the same location on Bridge 1374 as shown in Figure 57.

Figure 59. Photograph of largest delaminated area identified on Bridge 1333 in 2022.

4.4.3. Other Deterioration Observed

Note during the 2022 inspection, the superstructure and substructure of Bridge 1374 were observed to have large incipient spalls along the beams and pier caps. An example of their conditions is shown in Figure 60.

Figure 60. Cracks and spalls on the superstructure, deck beams, and pier cap of Bridge 1374 in 2022.

4.5. Summary of Results of Three-Year Field Investigation

Overall, the HFSTs appeared to be of good quality and generally in good condition. The skid numbers from 2020 indicate that they were providing good skid resistance despite the wear and overlay pop-out observed at the beginning of this study, and the latest skid numbers from 2023 show that the HFSTs are still of good quality in general. The skid numbers have reduced by about 13.8 % on average since 2020 across all the bridges measured in 2023. Spalling of the HFSTs was observed only in a few instances, either adjacent to reflective cracking around patch repairs where corrosion was ongoing or over very small areas on the order of a few square inches in size. Even though delaminations were identified, they appeared to be due to continued corrosion of the reinforcement in the deck, not due to disbondment of the HFST. The types of deterioration observed in the HFSTs in the field investigation were mainly consistent among all decks investigated and how they progressed over time is presented in the following discussion.

4.5.1. Abrasive Wear and Overlay Pop-Out

Wear due to traffic was present in all of the HFSTs except for those that were one year of age or less (Bridges 14 and 25 in 2020 and Bridge 1338 in 2022). The wear was characterized by fracture of the aggregates to the approximate level of the polymer binder and holes where aggregates or small agglomerations of aggregates that had not been fully adhered had been dislodged. With age, microscopic examination showed micro-cracking of the exposed binder resin and increasing open texture was caused not only by loss of aggregates but also by exposure of entrapped consolidation-type voids present below the top surface. In this context, the entrapped voids indicate an insufficient resin content or incomplete consolidation. The resin overlay must be placed at a moderately low resin content such that aggregate particles are boldly exposed to provide the high friction surface needed, but if resin is deficient, then aggregates will not be well-adhered.

Wear and aggregate loss was most severe in the wheel paths of the driving lane. The wheel paths of the passing lane tended to have less severe wear relative to the corresponding driving lane of the same deck based on visual observations. This coincides with the different skid numbers measured in the passing and driving lanes shown in Table 13 and Table 14, wherein the passing lanes consistently had a skid number higher than that of their corresponding driving lanes. The skid number of the passing lane was on average higher by 8.5 in 2020 and 13.1 in 2023. The shoulder areas generally appeared to be untouched with the surface aggregate particles mostly intact and well-adhered.

Of the HFSTs inspected, Bridges 1336, 1392, and 3734 had the least amount of wear observed. The relatively low wear observed on Bridge 3734 even after 4 years of traffic exposure is expected as it is a low-volume road (100 vehicles per day and only 3% truck traffic) and a low-speed overpass. The relatively low wear observed on Bridges 1336 and 1392 is unique as the HFSTs were 5 years and 4 years of age in 2022, respectively, and Bridge 1336 carries I-90 WB with 6,553 vehicles per day, of which 27% are trucks, and Bridge 1392 carries I-90 EB with 9,138 vehicles per day, of which 20% are trucks. However, it should be noted that only the passing lane of Bridge 1336 could be inspected in detail and the wear on the driving lane would be expected to be more severe.

4.5.2. Small Spalls in the HFST

Occasional small spalling of the overlay was observed on multiple bridges (Bridges 1670, 1682, 1367, 3734, and MM49.39). The spalls were on the order of one to two inches in diameter and typically there were only a few per bridge. Bridges 3734 and MM49.39 each had one spall, first seen in 2020, and Bridge 1367 had three spalls first seen in 2020 when the HFST was 4 years of age. Bridge 1682 had two spalls first identified in 2020 when the HFST was 5 years of age and no additional spalls were identified in the 2022 inspection. Bridge 1670 was the exception; four spalls were identified in 2020 and then dozens were observed in 2022 when the HFST was 7 years of age. The spalls in Bridge 1670 were typically located in the driving lane with some in the shoulder adjacent to the driving lane at the bridge approach.

While HFST spalls were observed on some bridges with HFSTs between 2 and 5 years of age, it is worth noting that Bridges 1333 and 1336 did not contain spalls in their inspected areas in 2022, when their HFSTs were 5 years of age. Further investigation would be required to determine the cause of the spalling on Bridge 1670, but the high density of spalls in the driving lane of Bridge 1670 is likely unusual rather than representative of a 7-year-old HFST. While the only other 7-year-old HFST to be investigated was on Bridge 1682, the lack of numerous spalls both on Bridge 1682 and in the passing lane and shoulders of Bridge 1670 after 7 years indicates that the HFST in the driving lane of Bridge 1670 was at relatively greater risk of loss of bond or localized spalling. This might be related to construction practices such as incomplete mixing of resin components or local areas of poor compaction. Further, Bridges 1682 and 1670 were inherently at greater risk of HFST spalls with time than the other bridges because no primer was used as part of the HFST system.

4.5.3. Transverse Cracking

Most of the HFSTs had transverse cracking that reflected from the deck. The exceptions were bridges whose HFSTs were one year old or less (Bridges 14 and 25 in 2020 and Bridge 1338 in 2022) and Bridge 3734, whose HFST was 4 years old but still did not have any visible cracks in 2022. Interestingly, transverse cracks appeared to develop progressively across the monitoring period for some bridges. The HFSTs on Bridges 14 and 25 did not have transverse cracks shortly after their construction in 2020, but developed regularly-spaced cracks within 2 years by 2022. Bridge 1392 did not have any visible transverse cracks in 2020 but transverse cracks were observed in 2022 in select locations, likely over the bents. And in 2020, Bridge 1428 had transverse cracks over the bents but had developed more regularly-spaced cracks by 2022.

4.5.4. Reflective Cracking at Patch Repairs

Reflective cracking around repair patch perimeters was prevalent in both of the Billings bridges. Reflective cracking of patches commonly occurs due to the use of patch materials with relatively high shrinkage, patch materials that are thermally incompatible with the concrete substrate, or an insufficient curing and drying period between patch repair and HFST installation. Based on the installation year of the HFSTs and the MDT’s records, it appears likely that the HFSTs were installed on the Billings bridges when a polymer-based patch material was still in use and the reflective cracking is that reported by the MDT as the reason they switched to a cementitious patch material.

Reflective cracking around patch perimeters did not appear to be an issue in the Missoula bridges, which reportedly used cementitious repair materials where possible and Sure Patch, an epoxy repair mortar kit by Dayton Superior, when traffic constraints required minimal disruptions. However, transverse cracking did coincide with the locations of full-depth repairs in Bridge 1367. The transverse cracks tended to appear across the top area of the full-depth repairs instead of around their perimeters, as was noted in 2020 and observed again in 2022. Bridge 1333 also reportedly had patch repairs placed just 4 days prior to HFST installation. While the HFST on Bridge 1333 had numerous cracks in the 2022 inspection, none of the cracks were obviously associated with patch perimeters.

4.5.5. Delaminations and Unsound Areas

The two Billings bridges (Bridges 1670 and 1682) had many delaminations in 2022, typically located at reflective cracks around patch repairs. Bridge 1682 additionally had three unsound areas that were not related to patch repairs or overlay edges and corners located in the shoulder. The two Missoula bridges that underwent detailed inspections (Bridges 1459 and 1367) did not have any delaminations or unsound areas as of 2022, but other bridges in the Missoula District, including Bridges 1333 and 3734, had numerous delaminations. However, when the HFST was chipped away over areas identified as delaminated on Bridge 3734, the HFST was found to be well-bonded such that the delaminations were likely due to continued corrosion of the reinforcing steel and not loss of bond of the HFST. The delaminations found in the other HFSTs are likely also due to continued corrosion rather than disbondment of the HFST, with the exception of a few delaminations at the edges and corners of the HFSTs.

4.5.6. Edge Damage of HFSTs

The HFSTs were typically cracked or delaminated at the approach edges at the bridge ends, likely due to snowplow impact and traffic abrasion. In some cases, such as Bridge 1459, the HFST at the bridge end had a spall, and in other cases, such as Bridge 1682, areas of the HFST edge had been worn fully away. The edges and corners of the HFSTs are known to be prone to higher stress due to thermal cycling; however, evidence of thermal incompatibility of the overlays was not found. The wear damage at approaches appears to be largely from snow plows.

5. LABORATORY INVESTIGATION OF HFSTS IN MONTANA

A laboratory investigation was conducted on cores taken from the four bridges that underwent detailed investigation. The HFST systems used on the four bridges are summarized in Table 18 for reference. The purpose of the laboratory investigation was to characterize the condition of the HFSTs, the protection they offered to the decks with respect to chloride penetration, and the durability and deterioration of the HFSTs.

Table 18. Description of High Friction Surface Treatments for the Bridges Chosen for Detailed Investigation

Bridge ID and Region	HFST: Year of Installation	HFST System Description			Partial-Depth (Class A) Repairs	Full-Depth (Class B) Repairs
		Primer	Polymer	Aggregates		
1670 Billings	2015	none	Pro-Poxy Type III D.O.T.	Armorstone	HD 50 & conventional concrete	none
1682 Billings	2015	none	Pro-Poxy Type III D.O.T.	Armorstone	none	conventional concrete
1459 Missoula	2018	Pro-Poxy 45	Pro-Poxy Type III D.O.T.	Lake Ranch Pit	Sure Patch	none
1367 Missoula	2016	Pro-Poxy 45	Pro-Poxy Type III D.O.T.	Lake Ranch Pit	present; material not recorded	conventional concrete

5.1. Core Sampling

WJE personnel collected cores with a nominal diameter of 3-5/8 to 3-7/8 inches from Bridges 1670, 1682, 1459, and 1367 during the 2020 and 2022 inspections. The cores from the 2020 inspection are labelled with the bridge number followed by a numeric core ID, e.g., 1459-1, and the cores from the 2022 inspection are labelled with the bridge number followed by an alphabetical core ID, e.g., 1459-A. The dates when the cores were taken are listed in Table 19 and the core locations are shown in the field notes in Appendix D. Cores were removed from the shoulder, driving, and passing lanes and from both intact and worn, cracked, or otherwise distressed areas in order to collect representative samples to assess the performance of the HFSTs. Table 20, Table 21, Table 22, and Table 23 summarize the locations and features of the cores collected from Bridges 1670, 1682, 1459, and 1367, respectively, in 2020, and Table 24 through Table 27 present the cores collected in 2022. In the tables, PDR refers to a partial-depth repair and FDR refers to a full-depth repair. Upon collection, the cores were shipped or driven to WJE's laboratory in Northbrook, Illinois.

Table 19. Bridge Inspection and Coring Dates

Bridge ID	2020 Inspection		2022 Inspection	
	Date Cored	No. of Cores	Date Cored	No. of Cores
1670	8/28/2020	9	9/12/2022	4
1682	8/28/2020	9	9/12/2022	5
1459	8/26/2020	10	9/13/2022	8
1367	8/24/2020	12	9/15/2022	7

Table 20. Description of Cores Sampled from Bridge 1670 in 2020

Core ID	Core Location	Field Notes	Diameter (in.)	Length (in.)	HFST Thickness (in.)	Presence of Repairs	Presence of Cracking	Reinforcing Steel	Cover (in.)[1]
1670-1	Shoulder	Good area	3.625	4.25 to 5.25	3/8 to 1/4	none	none	none	n/a
1670-2	Shoulder	Good area	3.625	2.625 to 3.25	1/4	none	none	none	n/a
1670-3	Shoulder	PDR; Reflective crack; Broke overlay during removal	3.625	Up to 3.375	1/4	Polymer PDR	Core was fractured at repair edge	none	n/a
1670-4	Driving Lane	PDR; fractured at repair/substrate plane during removal	3.625	3.25 to 4	3/16 to 1/4	Polymer PDR	none	none	2.25
1670-5	Driving Lane (Wheel Path)	--	3.625	5 to 5.625	1/8 to 1/4	none	none	none	5.25
1670-6	Driving Lane (Wheel Path)	PDR	3.625	3.25 to 3.75	3/16	Polymer PDR	none	One No. 7 bar	2
1670-7	Driving Lane (Wheel Path)	PDR; Reflective crack; Sediment build-up	3.625	0.625 to 2.5	1/8 to 1/4	Polymer PDR	Vertical crack between repair and substrate; extended through full depth of core	One No. 5 bar	1.75
1670-8	Driving Lane (Wheel Path)	Fractured during removal	3.625	0.375 to 1.5	1/8 to 3/16	none	none	none	n/a
1670-9	Driving Lane (Ctr Wheel Path)	--	3.625	3.375 to 4	1/8 to 3/16	none	Vertical crack present but did not reflect through HFST	none	n/a

Notes: [1] *Cover was measured to the top of the substrate/bottom of the HFST. In some cases, the concrete cover was able to be identified based on the imprint of rebar on the core even though a rebar was not present.*

Table 21. Description of Cores Sampled from Bridge 1682 in 2020

Core ID	Core Location	Field Notes	Diameter (in.)	Length (in.)	HFST Thickness (in.)	Presence of Repairs	Presence of Cracking	Reinforcing Steel	Cover (in.)[1]
1682-1	Shoulder	Good area	3.625	4.75 to 5.875	3/16 to ¼	none	Vertical crack; does not reach top surface of substrate or reflect through overlay	One No. 6 bar	2.25
1682-2	Driving Lane (Wheel Path)	--	3.6875	4.375 to 5.375	3/16	none	Vertical hairline crack, 1.375 inches deep	One No. 6 bar	1.875
1682-3	Shoulder	Good area	3.6875	4.75 to 5.375	¼ to 5/16	none	none	One No. 6 bar	2.0
1682-4	Shoulder	PDR	3.6875	4.75 to 5.75	¼	Polymer PDR	none	Top: No. 6 Bot.: No. 5	Top: 1.625 Bot.: 4.625
1682-5	Shoulder	PDR edge; Reflective crack	3.6875	1.25 to 3.625	1/8 to 5/16	Polymer PDR	Vertical cracking at repair edges; reflected in overlay	none	n/a
1682-6	Driving Lane (Wheel Path)	Delam in original deck adjacent to patch; Reflective crack; PDR	3.6875	1.5 to 2.0	3/16 to 5/16	Polymer PDR	Vertical, full-depth crack; reflected in overlay; not located at deck/repair interface	none	1.5
1682-7	Driving Lane (Wheel Path)	--	3.6875	4.5 to 5.0	¼ to 3/8	none	Several vertical cracks	One No. 6 bar	2.0
1682-8	Driving Lane	Crack; Sediment at delam planes between patch/deck and in original deck adjacent to patch	3.6875	2.125 to 3.75	¼ to 3/8	Polymer PDR	Vertical crack; located at deck/repair interface; reflected through overlay	none	n/a
1682-9	Driving Lane (Wheel Path)	PDR; Delam plane between patch/deck with sediment buildup	3.6875	3.625 to 4.75	5/16	Polymer PDR	none	One No. 6 bar	2.5

Notes: [1] *Cover was measured to the top of the substrate/bottom of the HFST. In some cases, the concrete cover was able to be identified based on the imprint of rebar on the core even though a rebar was not present.*

Table 22. Description of Cores Sampled from Bridge 1459 in 2020

Core ID	Core Location	Field Notes	Diameter (in.)	Length (in.)	HFST Thickness (in.)	Presence of Repairs	Presence of Cracking	Reinforcing Steel	Cover (in.)[1]
1459-1	Btwn Driving & Passing Lanes	At overlay lap; Concrete crack	3.6875	5.375 to 5.625	3/16 to 5/16	none	Vertical crack nearly full-depth of core; appears filled to 1.25 to 1.625 inches deep; not reflected in overlay	none	5.375
1459-2	Shoulder	Good area	3.6875	5.0 to 5.75	5/16	none	none	none	n/a
1459-3	Driving Lane (Wheel Path)	--	3.6875	4.875 to 5.375	5/16	none	none	none	5.375
1459-4	Shoulder	Good area	3.6875	5.375 to 5.625	1/4 to 5/16	none	none	none	n/a
1459-5	Shoulder	Good area	3.6875	5.375 to 6.25	1/4 to 5/16	none	none	none	n/a
1459-6	Wheel Path	--	3.6875	5.125 to 5.5	1/4	none	none	none	n/a
1459-7	Driving Lane (Btwn Wheel Paths)	Asphalt patch over HFST	3.6875	5.125 to 5.625	3/16 to 5/16	Asphalt overlaid on half of core	none	none	n/a
1459-8	Driving Lane (Center Wheel Path)	Overlay delam during core removal	3.6875	5.375 to 5.875	1/4	none	Horizontal crack in substrate near overlay	none	5.625
1459-9	Passing Lane (Wheel Path)	--	3.6875	5.125 to 5.625	1/4	none	none	none	n/a
1459-10	Passing Lane (Wheel Path)	--	3.6875	5.0 to 5.5	1/4	none	none	none	5.0

Notes: [1] *Cover was measured to the top of the substrate/bottom of the HFST. In some cases, the concrete cover was able to be identified based on the imprint of rebar on the core even though a rebar was not present.*

Table 23. Description of Cores Sampled from Bridge 1367 in 2020

Core ID	Core Location	Field Notes	Diameter (in.)	Length (in.)	HFST Thickness (in.)	Presence of Repairs	Presence of Cracking	Reinforcing Steel	Cover (in.)[1]
1367-1	Driving Lane (Wheel Path)	--	3.6875	5.625 to 6.125	1/8 to 1/4	none	Vertical cracking; does not reach substrate surface or reflect through overlay	none	n/a
1367-2	Shoulder	Good area	3.6875	5.625 to 6.0	3/16	none	none	none	6.0
1367-3	Driving Lane	Overlay spall	3.6875	6.0	1/8 to 3/16	none	Small vertical cracks at interstitial zones	none	n/a
1367-4	Driving Lane	Overlay spall	3.6875	5.25 to 5.75	1/8 to 1/4	none	none	none	n/a
1367-5	Driving Lane	PDR; Reflective crack	3.6875	5.5 to 6.125	3/16 to 1/4	Cementitious PDR	Horizontal cracking at HFST-substrate interface and PDR-substrate interface	none	n/a
1367-6	Driving Lane (Wheel Path	FDR	3.6875	5.5 to 5.875	3/16	FDR	none	none	n/a
1367-7	Shoulder	Good area	3.6875	3.125 to 3.875	3/16 to 3/8	none	none	none	3.5
1367-8	Driving Lane (Wheel Path)	Crack/Repair	3.6875	5.0 to 5.375	1/8 to 1/4	FDR	Diagonal cracking caused core to fragment	none	n/a
1367-9	Driving Lane (Wheel Path)	Crack/Repair	3.6875	4.75 to 5.75	3/16 to 1/4	FDR	Vertical crack at repair boundary; 2.5 inches deep	none	n/a
1367-10	Shoulder	--	3.6875	5.125 to 5.625	1/4 to 5/16	none	Vertical hairline crack; 3 inches deep; reflects through overlay	none	5.5
1367-11	Driving Lane (Center Wheel Path)	--	3.6875	5.25 to 5.75	3/16 to 1/4	none	none	none	5.5
1367-12	Driving Lane (Center Wheel Path)	--	3.6875	5.375 to 5.625	3/16	none	none	none	5.375

Notes: [1] *Cover was measured to the top of the substrate/bottom of the HFST. In some cases, the concrete cover was able to be identified based on the imprint of rebar on the core even though a rebar was not present.*

Table 24. Description of Cores Sampled from Bridge 1670 in 2022

Core ID	Core Location	Field Notes	Diameter (in.)	Length (in.)	HFST Thickness (in.)	Presence of Repairs	Presence of Cracking	Reinforcing Steel	Cover (in.)[1]
1670-A	Driving Lane (Wheel Path)	--	3.875	5.0 to 5.25	1/8 to 1/4	none	Minor horizontal cracking at HFST-substrate interface	One No. 6 bar	1.875
1670-B	Driving Lane (Wheel Path)	--	3.875	4.75 to 5.5	3/16 to 1/4	none	none	One No. 6 bar	1.875
1670-C	Shoulder	--	3.875	4.5 to 5.25	3/16 to 1/4	none	Horizontal cracking at HFST-substrate interface	One No. 6 bar	2.25
1670-D	Shoulder	--	3.875	2.5 to 3.75	3/16 to 1/4	none	Minor horizontal cracking at HFST-substrate interface	One No. 6 bar	2.325

Notes: [1] *Cover was measured to the top of the substrate/bottom of the HFST. In some cases, the concrete cover was able to be identified based on the imprint of rebar on the core even though a rebar was not present.*

Table 25. Description of Cores Sampled from Bridge 1682 in 2022

Core ID	Core Location	Field Notes	Diameter (in.)	Length (in.)	HFST Thickness (in.)	Presence of Repairs	Presence of Cracking	Reinforcing Steel	Cover (in.)[1]
1682-A	Driving Lane	Not on wheel path, sound area	3.875	4.25 to 5.5	1/8 to 3/16	none	Minor horizontal crack at HFST-substrate interface extending from void in HFST at interface	One No. 6 bar	1.875
1682-B	Shoulder	Sound area	3.875	5.0 to 5.5	3/16 to 3/8	none	none	One No. 6 bar	2.25
1682-C	Shoulder	Sound area	3.875	4.5 to 5.25	¼ to 3/8	none	none	One No. 6 bar	1.75
1682-D	Driving Lane	Sound area on top of patch	3.875	4.0 to 4.75	1/8 to ¼	Polymer PDR	none	One No. 6 bar	1.25
1682-E	Driving Lane	Sound area adjacent to patch	3.875	4.0 to 4.5	1/8 to ¼	none	Minor horizontal cracking at HFST-substrate interface	One No. 6 bar	1.25

Notes: [1] *Cover was measured to the top of the substrate/bottom of the HFST. In some cases, the concrete cover was able to be identified based on the imprint of rebar on the core even though a rebar was not present.*

Table 26. Description of Cores Sampled from Bridge 1459 in 2022

Core ID	Core Location	Field Notes	Diameter (in.)	Length (in.)	HFST Thickness (in.)	Presence of Repairs	Presence of Cracking	Reinforcing Steel	Cover (in.)[1]
1459-A	Passing Lane	--	3.875	5.5 to 6.0	3/16 to 3/8	none	none	none	n/a
1459-B	Passing Lane	Centered on longitudinal crack	3.875	4.75 to 5.5	3/16 to 5/16	none	Vertical crack; reflects through overlay; 4.5 inches deep	One epoxy-coated, No. 4 bar	2.875
1459-C	Driving Lane (Wheel Path)	--	3.875	5.0 to 5.75	¼ to 5/16	none	Horizontal cracking in substrate just underneath overlay	none	4.75
1459-D	Driving Lane (Wheel Path)	--	3.875	4.75 to 5.5	3/16 to 1/4	none	none	none	4.75
1459-E	Driving Lane (Wheel Path)	--	3.875	5.0 to 5.5	3/16 to 1/4	none	none	none	n/a
1459-F	Driving Lane (Wheel Path)	--	3.875	5.0 to 5.75	1/4	none	none	none	n/a
1459-G	Shoulder	Sound area	3.875	5.0 to 5.5	3/16 to 3/8	none	Horizontal cracking at and underneath HFST-substrate interface	none	n/a
1459-H	Shoulder	Sound area	3.875	4.75 to 5.25	3/16 to 3/8	none	none	none	n/a

Notes: [1] *Cover was measured to the top of the substrate/bottom of the HFST. In some cases, the concrete cover was able to be identified based on the imprint of rebar on the core even though a rebar was not present.*

Table 27. Description of Cores Sampled from Bridge 1367 in 2022

Core ID	Core Location	Field Notes	Diameter (in.)	Length (in.)	HFST Thickness (in.)	Presence of Repairs	Presence of Cracking	Reinforcing Steel	Cover (in.)[1]
1367-A	Passing Lane (Wheel Path)	Transverse crack; broke on removal	n/a	2.75 to 5.5	3/16 to 1/4	none	Transverse crack was over/beyond bar; HFST on one half of core broke off during removal	One epoxy-coated, No. 5 bar	2.625
1367-B	Passing Lane (Wheel Path)	Sound area	3.875	5.5 to 6.0	1/8 to 1/4	none	none	none	n/a
1367-C	Driving Lane (Wheel Path)	--	3.875	5.25 to 5.75	3/16 to 1/4	none	none	none	5.125
1367-D	Driving Lane (Wheel Path)	--	3.875	5.0 to 5.5	1/8 to 1/4	none	Horizontal cracking at HFST-substrate interface	none	n/a
1367-E	Driving Lane (Wheel Path)	Transverse crack; broke on removal	n/a	2.25 to 4.5	3/16 to 1/4	FDR	Transverse crack was over bar; very minor horizontal crack in substrate below HFST	One epoxy-coated, No. 5 bar	3.75
1367-F	Shoulder	--	3.875	4.75 to 5.25	3/16 to 1/4	none	Horizontal cracking at HFST-substrate interface	none	n/a
1367-G	Shoulder	--	3.875	4.75 to 5.25	1/8 to 1/4	none	Horizontal cracking in substrate just underneath overlay	none	n/a

Notes: [1] *Cover was measured to the top of the substrate/bottom of the HFST. In some cases, the concrete cover was able to be identified based on the imprint of rebar on the core even though a rebar was not present.*

5.2. Test Methods and Program

Select cores were subjected to the following tests.

5.2.1. Pavement Macrotexture Depth (Modified ASTM E965)

ASTM E965, *Standard Test Method for Measuring Pavement Macrotexture Depth Using a Volumetric Technique*, was performed to determine the average depth of a pavement's surface macrotexture. It is typically conducted in the field but was adapted for this project as a laboratory method for analysis of the cores. The modified ASTM E965 method was conducted on all cores that did not have a surface-breaking crack reflecting through the HFST.

The modified procedure consists of cleaning the test surface using compressed air and a soft-bristled brush to remove any visible residue or debris; care was taken not to dislodge the aggregates. A natural silica sand (standard Ottawa Sand, graded to pass a No. 20 sieve and provided by Humboldt) was placed on top of the core and carefully spread with a rubber, disk-shaped tool to fill the surface voids and create a smooth, flush surface with the tips of the aggregate particles. Sample photographs of a core before and after filling the voids are shown in Figure 61 and Figure 62, respectively. The mass of the sand required to fill the surface was measured and converted to volume using the bulk density, which was determined according to ASTM C29, *Standard Test Method for Bulk Density ("Unit Weight") and Voids in Aggregate*. The average pavement macrotexture depth, MTD, was calculated according to the standard. The voids located along the edge of the cores such that they could not hold sand were measured and subtracted from the core area used in the analysis. After testing was complete, the cores were cleaned in preparation for further testing.

Figure 61. Photograph of Core 1670-4 before undergoing modified ASTM E965 testing.

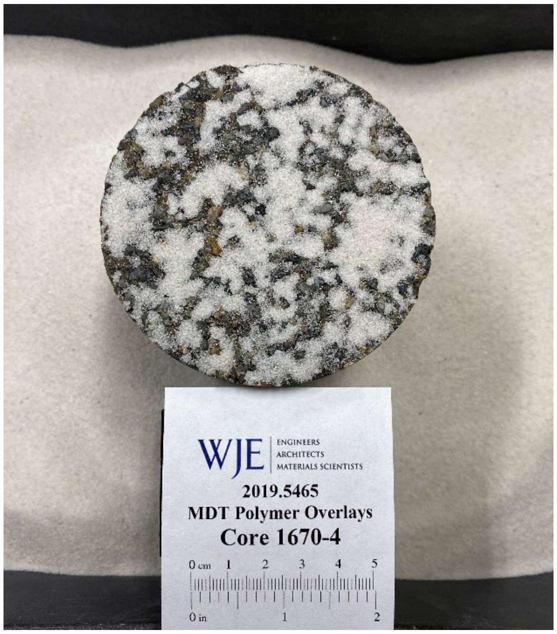


Figure 62. Photograph of Core 1670-4 after the surface has been filled with sand.

5.2.2. Petrographic Examination (ASTM C856)

Microscopical examination was conducted on ten select cores from 2020 to assess the characteristics of the overlay and substrate concrete, particularly the qualities of the resin and aggregates in the HFSTs, the bond between the HFST and the decks, and the crack characteristics. The microscopical examinations were conducted in general accordance with ASTM C856, *Standard Practice for Petrographic Examination of Hardened Concrete*, and observations were mainly made on the as-received cores and lapped cross sections of five of the cores selected for microscopical examination.

5.2.3. Rapid Chloride Penetration (ASTM C1202)

Testing was conducted in general accordance with ASTM C1202, *Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration*, on two to four cores from each sampled bridge with a total of eleven cores tested. All of the cores tested were collected in 2020. The cores were kept in a standard moist curing environment at approximately 73.5 ± 3.5°F and at least 95 percent relative humidity to ensure they were saturated prior to conditioning in preparation for the test. Test samples were cut from the top of the cores. Because the purpose of this investigation is to assess the electrical conductivity of the polymer topping and concrete, the HFST was not removed from the samples. Testing commenced in accordance with ASTM C1202.

5.2.4. Rapid Chloride Migration Test (AASHTO T 357)

Testing was conducted in general accordance with AASHTO T 357, *Standard Method of Test for Predicting Chloride Penetration of Hydraulic Cement Concrete by the Rapid Migration Procedure*, on one to two cores from each sampled bridge in 2020 with a total of seven cores tested. The cores were kept in a standard moist curing environment at approximately 73.5 ± 3.5°F and at least 95 percent relative humidity to ensure they were saturated prior to conditioning in preparation for the test. Test samples were then cut from the top of the cores and the HFST was not removed from the samples. Testing commenced in accordance with AASHTO T 357.

5.2.5. Bond Strength Testing (ASTM C1583)

Testing was conducted in general accordance with ASTM C1583, *Standard Test Method for Tensile Strength of Concrete Surfaces and the Bond Strength or Tensile Strength of Concrete Repair and Overlay Materials by Direct Tension (Pull-off Method)*, on the eleven cores collected in 2020 that had been previously subjected to ASTM C1202 testing. After ASTM C1202 testing was complete, the test samples were permitted to dry under standard laboratory conditions. To facilitate direct tension testing and proper alignment of the specimens, the test samples were partially cored using a drill with a nominal inner diameter of 2 inches. The test samples, which were 1.875 inches thick, were cored to a depth of approximately 1 inch. Pucks with a nominal diameter of 2 inches were adhered to the cored surface with a viscous epoxy (JBWeld Kwikweld). A second puck with a nominal diameter of 3 inches was adhered to the cut surface of the test sample. Both pucks were drilled and tapped to facilitate mounting into the test machine and care was taken to align the threaded holes to facilitate direct tension loading parallel to the axis of the specimen. Testing was conducted in a universal testing machine (Satec, Model 120HVL) at a constant load rate of 5 ± 2 psi per second, in accordance with ASTM C1583. The test results were analyzed and interpreted in accordance with ASTM C1583.

5.2.6. Chemical Methods

The HFSTs of four cores collected in 2020, one from each bridge deck (Cores 1670-8, 1682-7, 1459-9, and 1367-4), underwent additional microscopic inspection and compositional characterization using Fourier transform infrared (FTIR) spectroscopy and differential scanning calorimetry (DSC). Their degradation behavior was also characterized by thermogravimetric analysis (TGA). These chemical methods are described summarily below.

- **FTIR Spectroscopy.** FTIR spectroscopy is used to identify molecular structures. It is most useful for the identification of polymers and other organic components, although information about inorganic components can also be obtained. Infrared radiation supplies sufficient energy to produce vibrational motion in molecules. The output of the analysis is a plot showing peaks of absorbance at energies associated with the vibrations of functional groups (that is, a defined combination of bonded atoms). The spectrum can be interpreted based on the functional groups detected by the FTIR analysis as well as comparison to a library of spectra of known materials. An FTIR spectrum is sometimes referred to as a molecular fingerprint. In FTIR-ATR (Attenuated Total Reflectance) analysis, a solid material is analyzed directly, and the resulting spectrum is a mixture of compounds present in the solid material. Advanced FTIR imaging capability is enabled by μ-ATR FTIR technique which runs indentation of a Germanium ATR crystal on the specified sample surface.
- **Differential Scanning Calorimetry.** DSC measures the difference between the heat flowing from a sample and an inert reference as a function of time and temperature. This difference occurs as a result of samples absorbing or releasing heat associated with physical transitions in materials such as melting, crystallization, and glass transition, as well as chemical reactions including curing, polymerization, dehydroxylation, thermal decomposition and other processes. DSC analysis allows detection of thermal effects of a material under different heating or cooling conditions in inert or oxidative atmospheres. During the analysis, the heat flow is continuously recorded and plotted as a heat flow curve or a thermogram. The DSC thermogram is quantitatively analyzed to detect endothermic or exothermic effects and determine maximum peak temperature, peak area for transition and reaction enthalpies, specific heat capacity, etc. DSC can be used to measure cure properties of polymers.
- **Thermogravimetric Analysis.** TGA is a technique to measure the change in the mass of a sample as it is heated or cooled within a specified temperature range or held at constant temperatures. The mass changes occurring in samples are associated with not only physical phenomena such as evaporation, absorption, adsorption, or desorption but also chemical phenomena including thermal decomposition, chemisorption, oxidation, or reduction. When a sample is heated, it may lose or gain weight; this produces a step in the TGA curve. The results of a TGA measurement are presented as a TGA thermogram where the mass of the tested specimen is plotted against temperature and/or time. The first derivative of the TGA curve, known as the differential thermogravimetric curve (DTG), is employed to show the rate at which the mass changes and determine inflection points. TGA is often performed as a method of quantifying the polymers present in the samples.

5.2.7. Summary of Testing Program

The testing conducted on each core is provided in Table 28. The testing programs for the 2020 cores from each bridge are summarized in Table 29 through Table 32. The cores are categorized based on their location and features (i.e., the presence of any vertical cracking that would affect the test results and repairs) and the number of cores from each category subjected to each test is identified. A similar table summarizing the number of 2022 cores from the bridge deck driving lanes, passing lanes, and shoulders that were subjected to modified ASTM E965 testing is provided in Table 33.

Table 28. Testing Program for Each Core

Core ID	ASTM E965 (Mod.)	ASTM C856	ASTM C1202	AASHTO T 357	ASTM C1583	Chemical Methods	Notes
Bridge 1670							
1670-1	yes	--	--	yes	--	--	Consumed
1670-2	yes	yes	yes	--	yes	--	Consumed
1670-3	--	--	--	--	--	--	Reserved
1670-4	yes	--	--	--	--	--	Reserved
1670-5	yes	--	--	yes	--	--	Consumed
1670-6	yes	--	--	--	--	--	Reserved
1670-7	--	--	--	--	--	--	Reserved
1670-8	yes	yes	--	--	--	yes	Consumed
1670-9	yes	--	yes	--	yes	--	Consumed
1670-A	yes	--	--	--	--	--	Consumed
1670-B	yes	--	--	--	--	--	Consumed
1670-C	yes	--	--	--	--	--	Consumed
1670-D	yes	--	--	--	--	--	Consumed
Bridge 1682							
1682-1	yes	--	--	yes	--	--	Consumed
1682-2	yes	--	yes	--	yes	--	Consumed
1682-3	yes	yes	yes	--	yes	--	Consumed
1682-4	yes	--	--	--	--	--	Reserved
1682-5	--	--	--	--	--	--	Reserved
1682-6	--	--	--	--	--	--	Reserved
1682-7	yes	yes	--	--	--	yes	Consumed
1682-8	--	--	--	--	--	--	Reserved
1682-9	yes	--	--	--	--	--	Reserved
1682-A	yes	--	--	--	--	--	Consumed
1682-B	yes	--	--	--	--	--	Consumed
1682-C	yes	--	--	--	--	--	Consumed
1682-D	yes	--	--	--	--	--	Consumed
1682-E	yes	--	--	--	--	--	Consumed
Bridge 1459							
1459-1	yes	yes	--	--	--	--	Consumed
1459-2	yes	--	yes	--	yes	--	Consumed
1459-3	yes	--	--	yes	--	--	Consumed

Core ID	ASTM E965 (Mod.)	ASTM C856	ASTM C1202	AASHTO T 357	ASTM C1583	Chemical Methods	Notes
1459-4	yes	yes	yes	--	yes	--	Consumed
1459-5	yes	--	--	yes	--	--	Consumed
1459-6	yes	yes	yes	--	yes	--	Consumed
1459-7	--	--	--	--	--	--	Reserved
1459-8	yes	--	--	--	--	--	Reserved
1459-9	yes	--	--	--	--	yes	Reserved
1459-10	yes	--	--	--	--	--	Reserved
1459-A	yes	--	--	--	--	--	Consumed
1459-B	--	--	--	--	--	--	Reserved
1459-C	yes	--	--	--	--	--	Consumed
1459-D	yes	--	--	--	--	--	Consumed
1459-E	yes	--	--	--	--	--	Consumed
1459-F	yes	--	--	--	--	--	Consumed
1459-G	yes	--	--	--	--	--	Consumed
1459-H	yes	--	--	--	--	--	Consumed
Bridge 1367							
1367-1	yes	--	yes	--	yes	--	Consumed
1367-2	yes	--	--	yes	--	--	Consumed
1367-3	--	yes	--	--	--	--	Consumed
1367-4	--	--	--	--	--	yes	Reserved
1367-5	yes	yes	--	--	--	--	Consumed
1367-6	yes	--	--	--	--	--	Reserved
1367-7	yes	--	yes	--	yes	--	Consumed
1367-8	--	--	--	--	--	--	Reserved
1367-9	--	--	--	--	--	--	Reserved
1367-10	yes	yes	yes	--	yes	--	Consumed
1367-11	yes	--	--	yes	--	--	Consumed
1367-12	yes	--	yes	--	yes	--	Consumed
1367-A	--	--	--	--	--	--	Reserved
1367-B	yes	--	--	--	--	--	Consumed
1367-C	yes	--	--	--	--	--	Consumed
1367-D	yes	--	--	--	--	--	Consumed
1367-E	--	--	--	--	--	--	Reserved
1367-F	yes	--	--	--	--	--	Consumed
1367-G	yes	--	--	--	--	--	Consumed

Table 29. Testing Program for Cores Collected from Bridge 1670 in 2020

Core Type[1]	No. of Cores	E965	C856	C1202	T 357	C1583	Reserved
DR	3	3	1	1	1	1	
SH	2	2	1	1	1	1	
DR.pdr	2	2					2
SH.pdr	1						1
DR.c.pdr	1						1
Totals	**9**	**7**	**2**	**2**	**2**	**2**	**4**

Notes: *[1] DR indicates the core is from a driving lane while SH indicates the core is from a shoulder. A ".c" indicates the core is cracked. A ".pdr" indicates the core has a partial-depth repair.*

Table 30. Testing Program for Cores Collected from Bridge 1682 in 2020

Core Type[1]	No. of Cores	E965	C856	C1202	T 357	C1583	Reserved
SH	2	2	1	1	1	1	
DR.c	2	2	1	1		1	
DR.pdr	1	1					1
SH.pdr	1	1					1
DR.c.pdr	2						2
SH.c.pdr	1						1
Totals	9	6	2	2	1	2	5

Notes: *[1] DR indicates the core is from a driving lane while SH indicates the core is from a shoulder. A ".c" indicates the core is cracked. A ".pdr" indicates the core has a partial-depth repair.*

Table 31. Testing Program for Cores Collected from Bridge 1459 in 2020

Core Type[1]	No. of Cores	E965	C856	C1202	T 357	C1583	Reserved
DR	3	2	1	1	1	1	1
SH	3	3	1	2	1	2	
PL	2	2					2
DR.c	2	2	1				1
Totals	10	9	3	3	2	3	4

Notes: *[1] DR indicates the core is from a driving lane while SH indicates the core is from a shoulder. PL indicates the core is from the passing lane. A ".c" indicates the core is cracked.*

Table 32. Testing Program for Cores Collected from Bridge 1367 in 2020

Core Type[1]	No. of Cores	E965	C856	C1202	T 357	C1583	Reserved
DR	4	3		2	1	2	1
SH	2	2		1	1	1	
DR.c	1		1				
SH.c	1	1	1	1		1	

Core Type[1]	No. of Cores	E965	C856	C1202	T 357	C1583	Reserved
DR.pdr	1	1	1				
DR.fdr	1	1					1
DR.c.fdr	2						2
Totals	**12**	**8**	**3**	**4**	**2**	**4**	**4**

Notes: [1]DR indicates the core is from a driving lane while SH indicates the core is from a shoulder. A ".c" indicates the core is cracked. A ".pdr" indicates the core has a partial-depth repair while a ".fdr" indicates the core as a full-depth repair.

Table 33. Modified ASTM E965 Testing Program for Cores Collected in 2022

Core Type[1]	Bridge 1670	Bridge 1682	Bridge 1459	Bridge 1367
DR	2	2	4	2
SH	2	2	2	2
PL			1	1
DR.pdr		1		

Notes: [1]DR indicates the core is from a driving lane while SH indicates the core is from a shoulder. PL indicates the core is from the passing lane. A ".pdr" indicates the core has a partial-depth repair.

5.3. Laboratory Test Results

The results of the laboratory testing conducted on Bridges 1670, 1682, 1459, and 1367 are presented by test method below.

5.3.1. Pavement Macrotexture Depth

5.3.1.1. 2020 Test Results

The mean texture depths measured for each 2020 core that was tested using the modified ASTM E965 test procedure are presented in Table 34. The raw data from this test is provided in Appendix E. The mean texture depths of the cores collected from the driving lanes and those from the shoulders are compared in Table 35. On average, 2020 cores taken from driving lanes showed a mean texture depth of 0.085 inches while cores taken from shoulders demonstrated a mean texture depth of 0.133 inches. The two cores taken from the passing lane of Bridge 1459 in 2020 had an average mean texture depth of 0.108 inches.

Table 34. Summary of Mean Texture Depths Measured for Each 2020 Core Tested Using Modified ASTM E965 Procedure

Core ID	Mean Texture Depth (in.)	Core ID	Mean Texture Depth (in.)
Bridge 1670		**Bridge 1682**	
1670-1	0.132	1682-1	0.126
1670-2	0.149	1682-2	0.087
1670-4	0.108	1682-3	0.123
1670-5	0.072	1682-4	0.130
1670-6	0.078	1682-7	0.063

Core ID	Mean Texture Depth (in.)	Core ID	Mean Texture Depth (in.)
1670-8	0.077	1682-9	0.067
1670-9	0.111		
Bridge 1459		**Bridge 1367**	
1459-1	0.127	1367-1	0.085
1459-2	0.110	1367-2	0.125
1459-3	0.089	1367-5	0.104
1459-4	0.160	1367-6	0.081
1459-5	0.151	1367-7	0.158
1459-6	0.075	1367-10	0.096
1459-8	0.049	1367-11	0.087
1459-9	0.116	1367-12	0.088
1459-10	0.108		

Table 35. Comparison Between Average Mean Texture Depths Measured in Driving Lanes (DR) and Shoulders (SH) in 2020

Bridge ID	MTD in DR (in.)	MTD in SH (in.)
1670	0.089	0.141
1682	0.073	0.126
1459	0.085	0.140
1367	0.089	0.126
Average MTD	**0.085**	**0.133**
Standard Deviation	0.019	0.020
No. of Datapoints	17	11
Maximum MTD	0.127	0.160
Minimum MTD	0.049	0.096

5.3.1.2. 2022 Test Results

The mean texture depths measured for each 2022 core that was tested using the modified ASTM E965 test procedure are presented in Table 36. The raw data from this test is provided in Appendix E. The mean texture depths of the cores collected from the driving lanes and those from the shoulders are compared in Table 37. On average, 2022 cores taken from driving lanes showed a mean texture depth of 0.061 inches while cores taken from shoulders demonstrated a mean texture depth of 0.153 inches.

Table 36. Summary of Mean Texture Depths Measured for Each 2022 Core Tested Using Modified ASTM E965 Procedure[1]

Core ID	Mean Texture Depth (in.)	Core ID	Mean Texture Depth (in.)
Bridge 1670		**Bridge 1682**	
1670-A (WP)	0.066	1682-A (WP)	0.060
1670-B (WP)	0.048	1682-B (SH)	0.133

Core ID	Mean Texture Depth (in.)	Core ID	Mean Texture Depth (in.)
1670-C (SH)	0.132	1682-C (SH)	0.124
1670-D (SH)	0.149	1682-D (WP)	0.079
		1682-E (WP)	0.081
Bridge 1459		**Bridge 1367**	
1459-A (WP)	0.074	1367-B (WP)	0.131
1459-C (WP)	0.071	1367-C (WP)	0.046
1459-D (WP)	0.066	1367-D (WP)	0.046
1459-E (WP)	0.058	1367-F (SH)	0.130
1459-F (WP)	0.052	1367-G (SH)	0.095
1459-G (SH)	0.228		
1459-H (SH)	0.234		

Notes: [1]*WP identifies a core as belonging to the wheel-path of either the driving or passing lane and SH identifies a cores as belonging to the shoulder.*

Table 37. Comparison Between Average Mean Texture Depths Measured in Driving Lanes and Shoulders in 2022

Bridge ID	MTD in DR (in.)	MTD in SH (in.)
1670	0.057	0.140
1682	0.073	0.128
1459	0.062	0.231
1367	0.046	0.112
Average MTD	**0.061**	**0.153**
Standard Deviation	0.012	0.050
No. of Datapoints	11	8
Maximum MTD	0.081	0.234
Minimum MTD	0.046	0.095

5.3.2. HFST Characteristics Based on Petrographic Examination

A summary of the key findings of the petrographic examination is provided below. The full petrographic report, including photographs from the examination, is provided in Appendix F. Cores 1670-8, 1682-7, 1459-1, 1367-3, and 1367-5 were examined in detail. The findings from the detailed examinations are as follows:

1. **Characteristics of the Overlay:**
    a. Each core consists of a sand-resin polymer overlay 0.2 to 0.3 inches thick and substrate concrete. The two layers of materials were generally well bonded.
    b. The sand-polymer overlay appeared to be similar overall among the cores studied. Minor differences were noticed in amounts of air voids, thickness, and possibly sand-resin ratios.
    c. Sand was generally similar among the cores. Sand in the overlay was mainly composed of various siliceous volcanic rocks and appeared to be dense and durable. However, fractures or microcracks

were frequently observed on exposed sand particles on the top surface. Sand particles in the overlay were frequently angular and occasionally near-elongated or near-flat.

   d. The sand-polymer overlay appeared to have performed well or satisfactorily. No major cracks or other forms of distress were observed in the overlay. No anomaly or unusual features were noticed.
   e. Vertical, hairline or thicker cracks were observed in the substrates of the five cores examined. The vertical cracks (or vertical joint in the case of Core 1367-5) generally did not appear to be reflected in the sand-polymer overlay.
   f. The sand-resin bond of the overlay was generally tight. Sockets left by dislodged or plucked sand particles were observed on the top surface but occurrences were infrequent. The top surface appeared to be skid-resistant overall.
   g. The resin/polymer binder appeared to be polished, smooth, clear, amber colored and somewhat brittle on the top surface due to exposure to traffic and weathering. The binder below the top surface at greater depth appeared to be milky, less transparent, and less brittle when tested by a steel pick.
   h. A crack sealer or the resin component of the overlay appeared to have penetrated to significant depths in the vertical cracks. The crack sealer appeared to be darker in color than the resin/polymer and possibly contained a filler material. Thin section examination or chemical analysis would be needed to assess the similarity or dissimilarity.
   i. The sand-polymer overlay contained varying amounts of entrapped air voids, which occurred as holes on the top surface and contributed to the surface roughness.
   j. Core 1367-3 exhibited localized scaling, spalling, or loss of the overlay. The surface loss was not observed in other cores.

2. **Characteristics of the Substrate Concrete:**
   a. The substrate concretes are mainly composed of siliceous gravel (nominal top size 1/2-inch) and natural siliceous sand dispersed in a well-air-entrained cementitious paste. Air voids are generally small, spherical, and abundant. The air-void system appeared to be adequate to protect the concrete from distress caused by cyclic freeze-thaw.
   b. The concrete is well consolidated and the distribution of aggregate, paste, and air voids appeared to be fairly uniform overall.
   c. No evidence of materials-related distress such as alkali-silica reaction or freeze-thaw damage was observed in the substrate concretes or the overlay.
   d. Substrate concrete was roughened or prepared to a CSP estimated at 3 to 5 in Core 1367-3 and 5 to 6 in Cores 1367-5 and 1670-8, respectively. Microcracks or bruising related to surface preparation appeared to be infrequent overall.

Cores 1670-2, 1682-3, 1459-4, 1459-6, and 1367-10 were examined as-received to preserve them for further testing. Brief examinations of the five as-received cores show that the overlay and the substrate appeared to be similar to the other five cores in overall composition. The top surfaces of these cores exhibited no to moderate traffic-related smoothening and erosion. No cracks were observed in the substrate or in the overlay based on brief visual examinations. The bond between the overlay and the substrate concrete appeared to be tight and intact in these cores.

5.3.3. Rapid Chloride Penetrability

All eleven of the cores subjected to ASTM C1202 testing demonstrated a charge passed of 0 Coulombs, indicating that the HFSTs are essentially nonconductive. These included cores from both the driving and shoulder lanes. Two of the cores (1682-2 and 1367-10) contained hairline or partial depth cracks. The crack in Core 1367-10 broke the surface of the core but was a hairline crack while the crack in 1682-2 did not reflect through the overlay.

5.3.4. Rapid Migration Testing

The presence of preexisting chloride contamination of the deck concrete makes rapid migration testing results difficult to interpret. The initial and final test parameters for the AASHTO T 357 testing are presented in Table 38 and the penetration depths are presented in Table 39. Full test reports are provided in Appendix E. Silver precipitates form when the silver nitrate solution reacts with chlorides in the test sample, resulting in a light, silver-tinted color where the chlorides are present. Of the cores subjected to AASHTO T 357 testing in this program, only core 1367-2 developed a traditional boundary between a light-colored, chloride-contaminated area and a darker, non-chloride-contaminated area (shown in Figure D.11), and at a relatively shallow average penetration depth of 4 millimeters. The test samples more typically demonstrated transitions between the original substrate color, a tan color, and a dark grey color (Figure D.12) likely due to reaction with the chlorides already existing in the deck cores. Based on prior WJE experience, carbonation of the bridge deck surface prior to placement of the overlay may have also interfered with the results of the AASHTO T 357 testing.

Table 38. Summary of Initial and Final Conditions During AASHTO T 357 Testing

Specimen ID	Initial Current at 60 V mA	Final Current at 60 V mA	Initial Temperature, °F		Final Temperature, °F	
			NaOH	NaCl	NaOH	NaCl
1670-1	0.6	3.7	71.2	73.8	71.6	71.8
1670-5	0.2	2.2	71.2	74.1	71.4	71.4
1682-1	1.6	5.0	71.1	74.5	71.4	71.4
1459-3	3.1	15.3	71.4	71.4	72.1	72.1
1459-5	0.4	1.7	71.8	71.4	72.0	72.1
1367-2	7.8	24.6	69.6	74.5	71.8	72.0
1367-11	16.2	21.8	70.0	73.4	75.0	75.2

Table 39. Measured Penetration Depths from AASHTO T 357 Testing[1]

Specimen ID	Penetration Depth, mm								Average Penetration mm	Rate of Penetration mm/V-h
	1	2	3	4	5	6	7	8		
1670-1[2]	9	0	*	6	*	5	7	4	5	0.005
	43	32	27	31	45	44	45	45	39	0.036
1670-5	47	47	32	28	29	31	35	45	37	0.034
1682-1	34	40	41	42	44	45	39	*	41	0.038
1459-3	45	45	16	*	*	34	37	32	35	0.032
1459-5	19	24	24	30	26	34	25	37	27	0.025

Specimen ID	Penetration Depth, mm								Average Penetration mm	Rate of Penetration mm/V-h
	1	2	3	4	5	6	7	8		
1367-2	5	3	5	6	4	2	0	*	4	0.003
1367-11	3	29	48	46	46	44	44	45	38	0.035

Notes: [1] *An "*" indicates that the measurement was obstructed by an aggregate. Note that no averages are shown due to the general lack of a distinct chloride front.*

[2] *The test sample from core 1670-1 demonstrated to locations of color change, resulting in two rows of penetration depths.*

Figure 63. Photograph of the split test sample from core 1367-2 after AASHTO T 357 testing. The red arrow identifies the boundary between the silver-tinted color, which indicates chloride contamination, and the non-chloride-contaminated substrate.

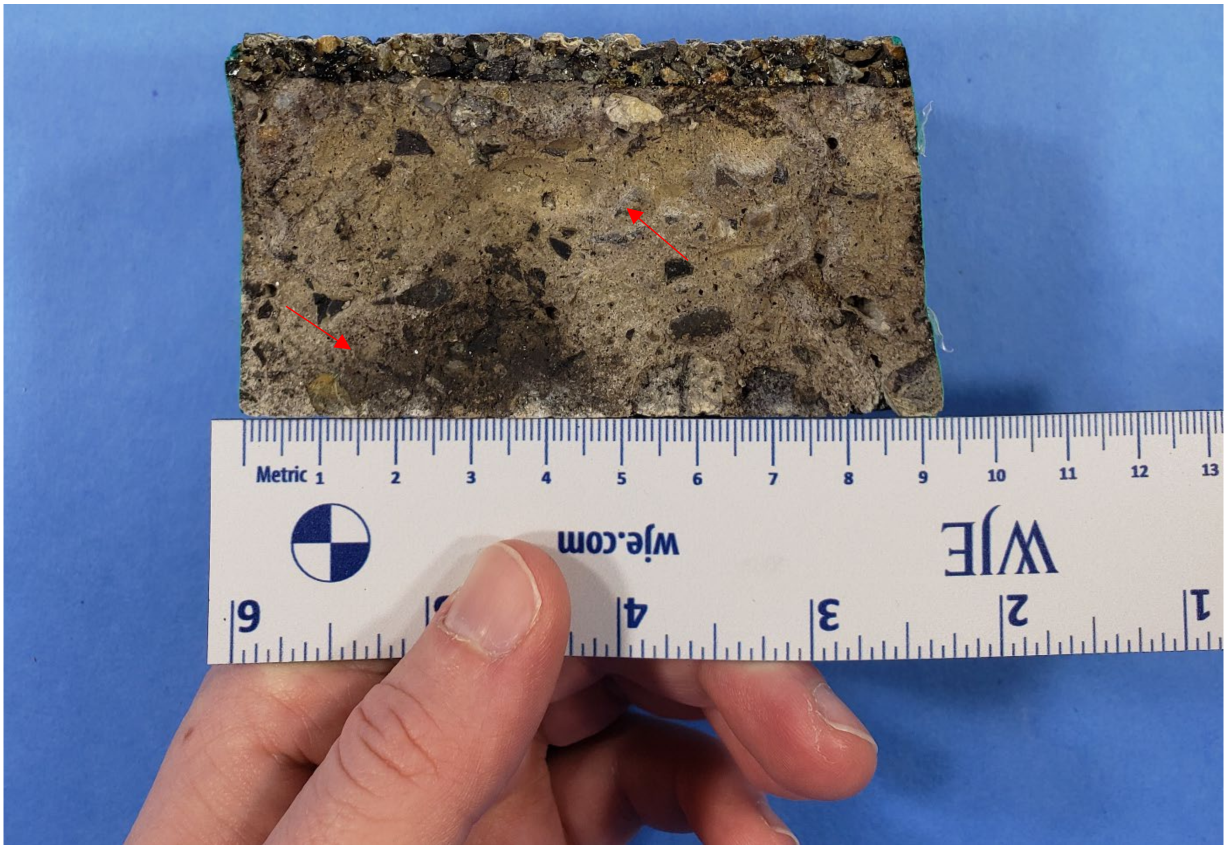


Figure 64. Photograph of the split test sample from core 1670-1 after AASHTO T 357 testing. The red arrows identify the color change lines.

The solution used in AASHTO T 357 testing was found to have highlighted the polymer matrix on the cores and so several of the test samples were inspected under the microscope. The examination revealed microcracking in the polymer of the HFST on Bridge 1367 (Figure 65). The polymer matrices of the HFSTs from Bridges 1670, 1682, and 1459 were relatively intact based on the inspection of their AASHTO T 357 cores. Interestingly, this correlates with the final currents measured in the test. The cores from Bridge 1367 had final currents of 24.6 and 21.8 mA while the final currents of the cores from the other three bridges were typically 5.0 mA or less, with the exception of Core 1459-3, which had a final current of 15.3 mA.

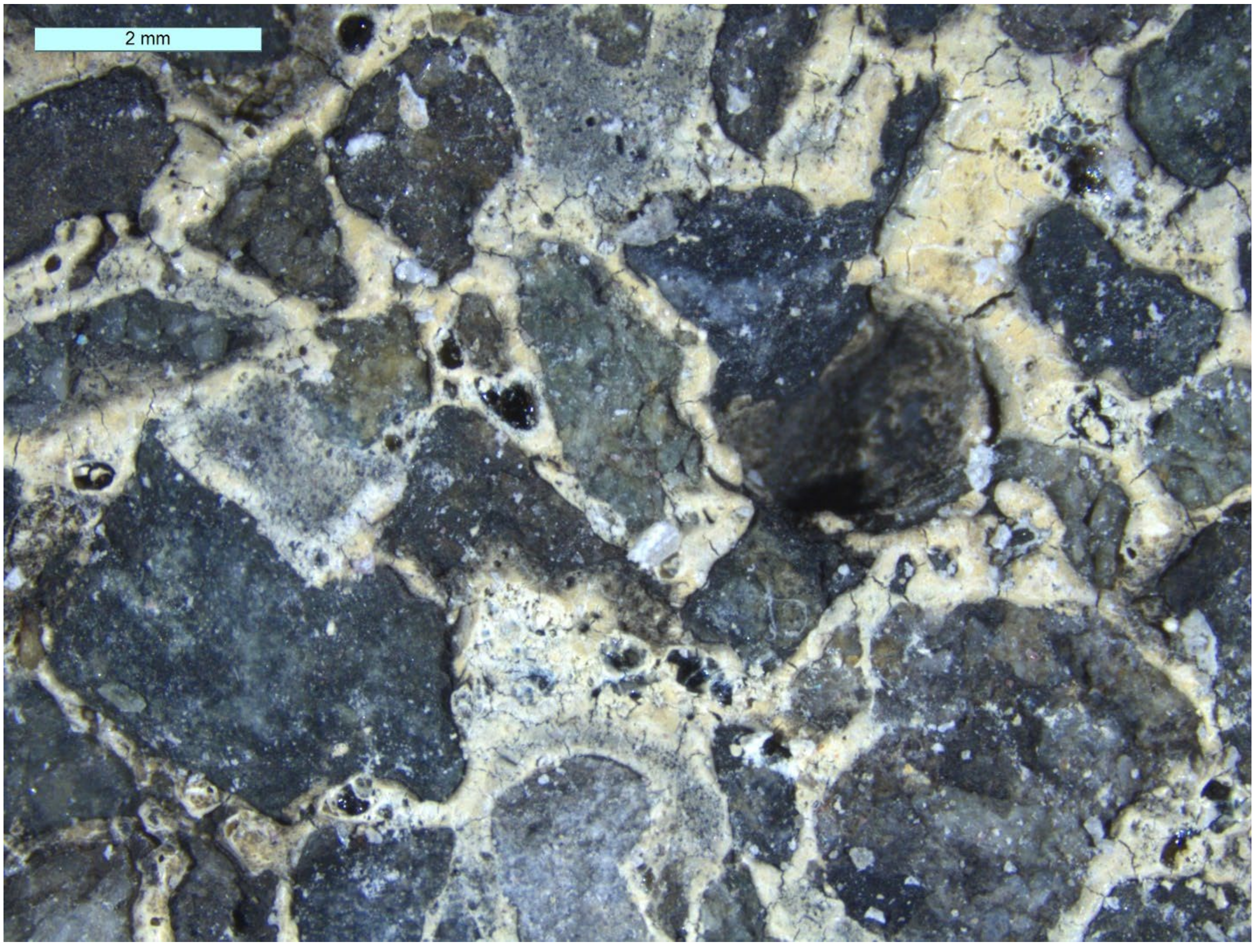


Figure 65. Photograph of the HFST on Core 1367-11 after AASHTO T 357 testing showing microcracks in the polymer matrix.

5.3.5. Bond Strength

The bond strengths measured for the eleven cores collected in 2020 and subjected to ASTM C1583 testing are listed in Table 40 as well as the type of fracture observed. The Billings bridges had slightly higher pull strengths of 459 psi for Bridge 1670 and 481 psi for Bridge 1682. Bridge 1459 in Missoula had an average pull strength of 422 psi. Bridge 1367 had a good to moderate average pull strength of 329 psi. All of the cores except for 1367-12 fractured in the deck substrate, indicating that the bond strength between the HFST and the deck substrate is typically stronger than the tensile strength of the deck substrate. For Core 1367-12, approximately 15 percent of the fracture area was at the interface between the HFST and 85 percent in the deck substrate, as shown in Figure 66.

Table 40. Results of ASTM C1583 Bond Strength Testing (2020)

Core ID	Max. Load	Bond Strength	Fracture Type
1670-2	1338 lb	426 psi	Deck substrate
1670-9	1545 lb	492 psi	Deck substrate
1670 Avg.	**1442 lb**	**459 psi**	**n/a**
1682-2	1433 lb	456 psi	Deck substrate

Core ID	Max. Load	Bond Strength	Fracture Type
1682-3	1591 lb	506 psi	Deck substrate
1682 Avg.	**1512 lb**	**481 psi**	**n/a**
1459-2	1255 lb	399 psi	Deck substrate
1459-4	1460 lb	465 psi	Deck substrate
1459-6	1266 lb	403 psi	Deck substrate
1459 Avg.	**1327 lb**	**422 psi**	**n/a**
1367-1	924 lb	294 psi	Deck substrate
1367-7	842 lb	268 psi	Deck substrate
1367-10	1238 lb	394 psi	Deck substrate
1367-12	1127 lb	359 psi	Mostly deck substrate; approximately 15% of area failed at interface between HFST and deck substrate
1367 Avg.	**1033 lb**	**329 psi**	**n/a**

Figure 66. Top view of fractured surface from testing of Core 1367-12 showing that while the majority of the fracture occurred in the deck substrate, a small area (approximately 15 percent) occurred at the interface between the HFST and the bridge deck.

5.3.6. Results of Chemical Testing

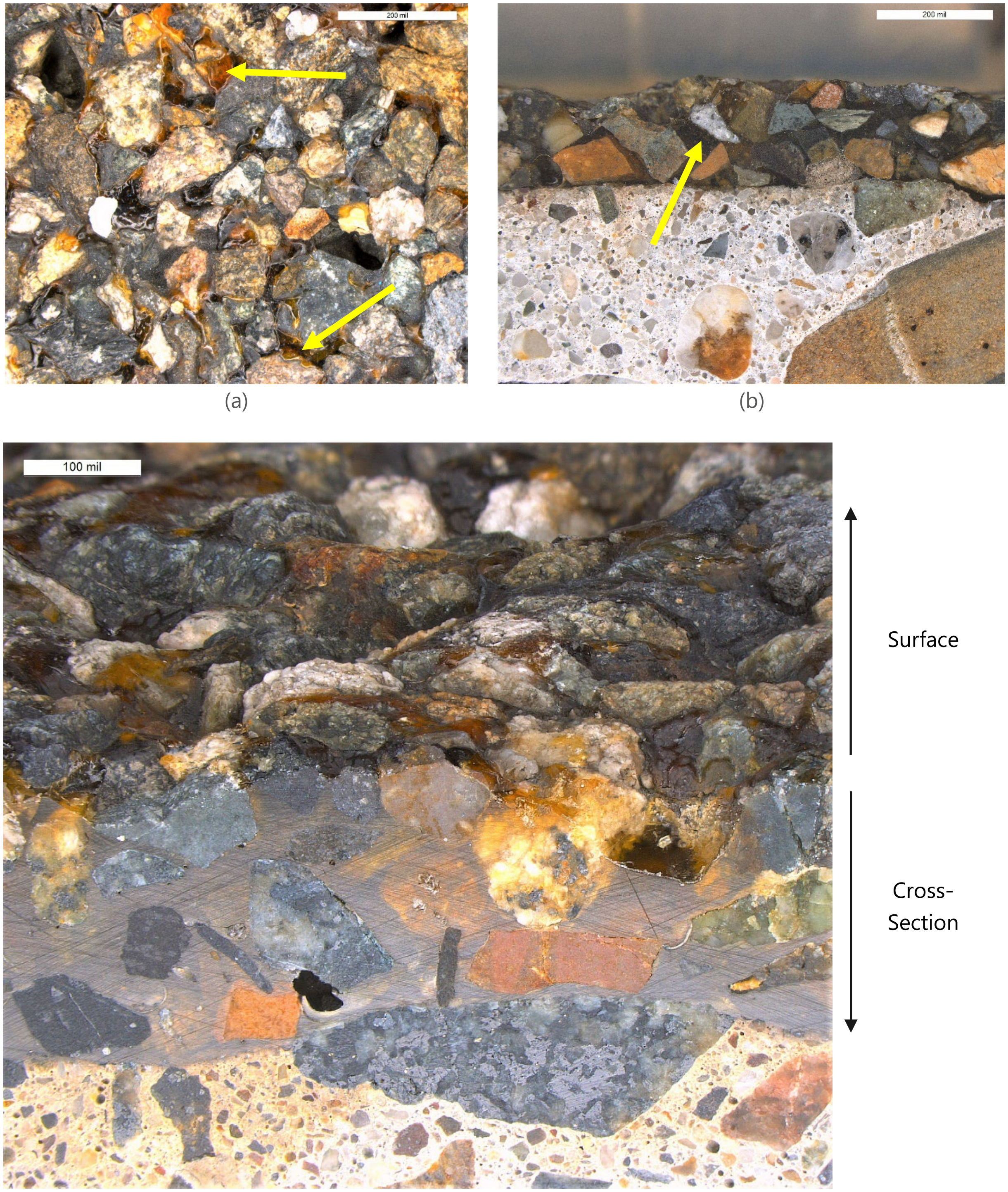
For each core subjected to chemical testing (1670-8, 1682-7, 1459-9, and 1367-4), a section from the top of the core consisting of the polymer overlay and a portion of the concrete substrate was cut from the larger core sample and then sectioned horizontally. One section was polished for microscopical evaluation (Figure 67) and the other section was pulverized for polymer collection (Figure 68). The clear and shiny, amber-colored material observed sporadically between aggregates in the plan view of the overlay (Figure 67(a)) is the polymer binder connecting aggregates as shown in the cross-sectional view of the overlay (Figure 67(b)). The connectivity of the polymer binder as a matrix and the core surface is presented well at a tilted angle using a Z-stack function of a microscope in Figure 67(c).

To obtain the polymer binders from the samples, the surface of the overlay on the second section from each core was pulverized using a chisel and hammer, producing agglomerates of the binders and the aggregates. They were further reduced to break and separate aggregates from the binder. During the process, sample 1682-7 was notably more brittle. In Figure 68, black specks are observed in the collected amber polymer binders even after brushing. The black specks are aggregate debris resulting from the pulverizing process. The presence of the aggregate debris affixed to the amber binder is indicative of the high strength of the bond between the aggregate and binder, as the aggregate and binder were not able to be separated cleanly along their adhesive interface.

5.3.6.1. Compositional Characterization by FTIR

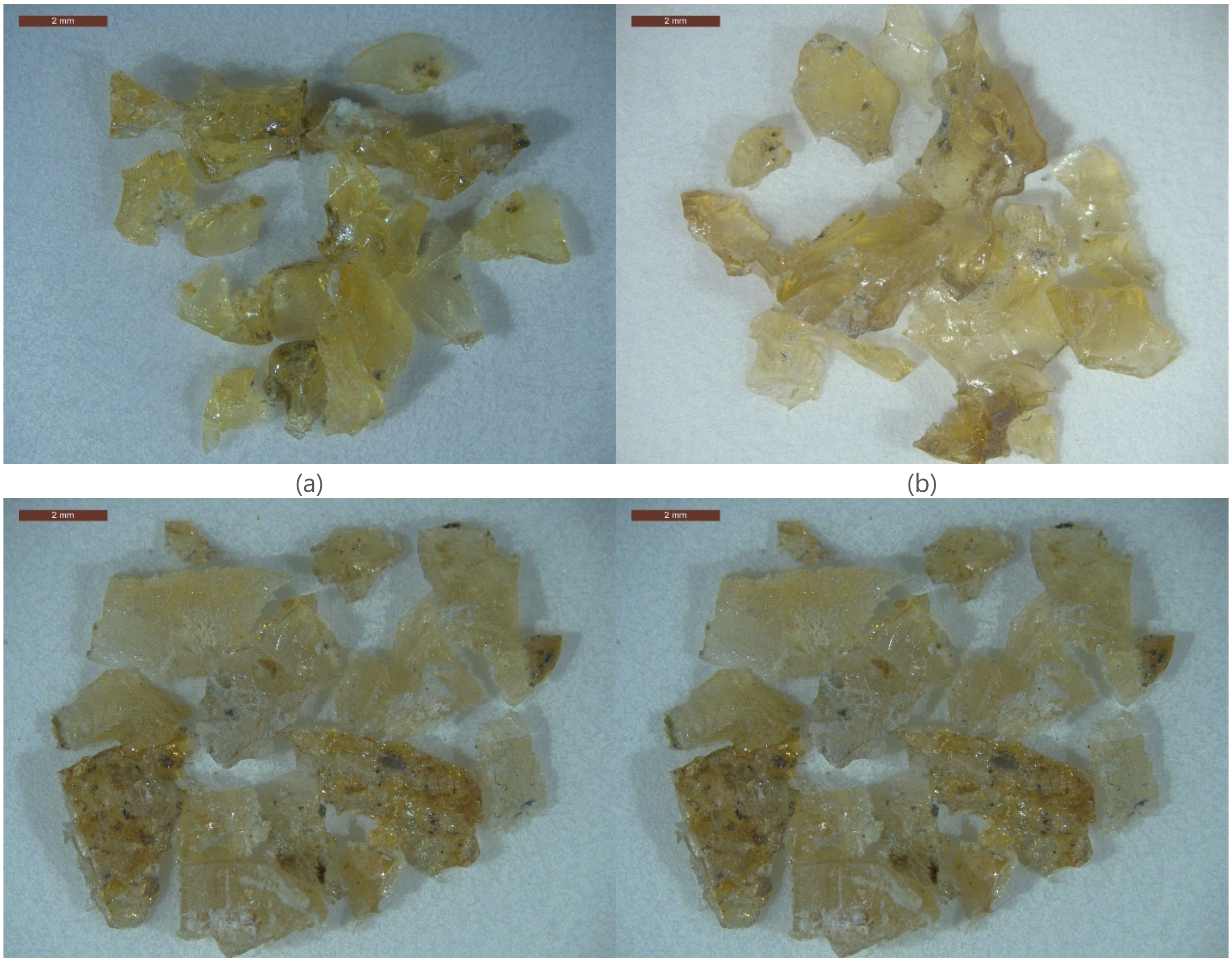
The isolated polymer binders were analyzed using an attenuated total reflectance (ATR) attachment for the FTIR, which allowed for direct analysis of solid samples. When possible, the samples were positioned such that areas free of black specks were measured. The spectra resulting from the analysis of the four selected samples were consistent with one another, indicating similar chemical compositions. The spectra most closely match a reference spectrum of epoxy resin.

To overcome the limitations of the sample preparation procedures, such as the inclusion of aggregates and random sampling within the overlay, an advanced technique of μ-ATR FTIR was introduced. The cross-section in Figure 67(b) was imaged by an FTIR microscope and the location of interest was specified. Then, the germanium ATR crystal was brought down to indent the specified location and an FTIR-ATR spectrum was collected. For each sample, four locations were measured in a 1.5-mm by 1.1-mm area. It was possible to measure not only aggregate-free locations in the polymer binder samples, but also locations at different depths within the overlay. Regardless of the samples and locations, similar spectra were obtained with characteristic peaks of solely epoxy resin epoxide groups.



(a) (b) (c)

Figure 67. Optical microscope images captured at 7.8X magnification: (a) surface and (b) cross-section of 1367-4. The yellow arrows indicate clear amber areas between aggregates, which are polymeric binding resins. To observe connectivity of the binding resin, 1682-7 was tilted and imaged using a Z-stack function in the photo (c) exhibiting both surface and cross section.



(a) (b)

(c) (d)

Figure 68. The binding polymer resins were isolated from aggregates. After the aggregates were crushed and brushed off, some aggregate material still remains in the binding resins. They are observed as black specks. (a) 1670-8, (b) 1682-7, (c) 1459-9, and (d) 1367-4. Captured at 16X magnification.

5.3.6.2. Thermal Analysis

TGA was performed to investigate the degradation behavior of the polymer binder. The isolated binder samples were heated to 600°C under nitrogen and then up to 950°C in air. This temperature program was intended to completely decompose any polymeric materials. Preliminary testing of collected aggregate materials showed that the aggregates did not show any noticeable mass change in the specified temperature range. As shown in Figure 69, all of the samples were observed to have residual mass in the inset of Figure 69, which is attributed to the aggregate specks remaining in the isolated binder samples. TGA thermograms were adjusted by calculating the true mass of the polymer binder based on the residual mass.

When comparing the thermograms of the four samples analyzed, a notable difference occurs during the mass change which begins at around 150°C and results in a mass reduction of approximately 30% by 350°C, beyond which the weight loss (%) curves merge. This divergence in mass loss was further investigated using the differential thermogravimetric curve (DTG), i.e., the first derivative of the TGA curve, shown in Figure 70. The first peak of the DTG curve showing the rate at which the mass changes was analyzed to determine peak area, which is the mass loss during the first reduction, and the onset temperature of the peak where the mass loss starts. The mass loss of 1682-7 is the smallest and delayed compared to the other samples as shown in Figure 70(b) and Figure 70(c).

To understand the difference in the degradation behavior of the polymer binders, the isolated binder samples were heated up to 300°C at a rate of 10°C per minute under nitrogen and analyzed by DSC. Two exothermic peaks were obtained as shown in Figure 71(a). The smaller peak between 100°C and 170°C (1st peak) is attributed to post-curing of the polymer binder that was not fully cured in service. The larger peak at temperatures beyond 200°C (2nd peak) is supposed to represent the portion of the chemical reaction that may not occur in normal service conditions. The peak area was measured and then normalized by the sample mass to determine the normalized exothermic heat. However, the uncertainty of the mass of the remaining aggregates makes sample-to-sample comparison difficult as shown in Figure 71(b). Thus, the exothermic heat related to post-curing was adjusted by normalizing the 1st peak area with the 2nd peak area as an internal reference. In Figure 71(c), 1682-7 shows a smaller ratio while 1367-4 has a larger ratio. In other words, 1682-7 was cured more and thus had a higher crosslinking density than 1367-4.

The crosslinked structure of the polymer binder assessed by DSC correlates well to the degradation behavior measured by TGA. As shown in Figure 72, the TGA-determined weight loss (%) shows a positive correlation to the exothermic heat ratio by DSC while the TGA-determined onset temperature exhibits a negative correlation. Less post-curing was observed when the polymer binder had experienced more complete curing in service. As the chemical structure of the polymer is highly crosslinked, it becomes more resistant to heat and its thermal decomposition is delayed. The highly crosslinked polymer binder of 1682-7 correlates well to the observation during sample preparation that it was hard but brittle and showed less elasticity and easy separation from aggregates.

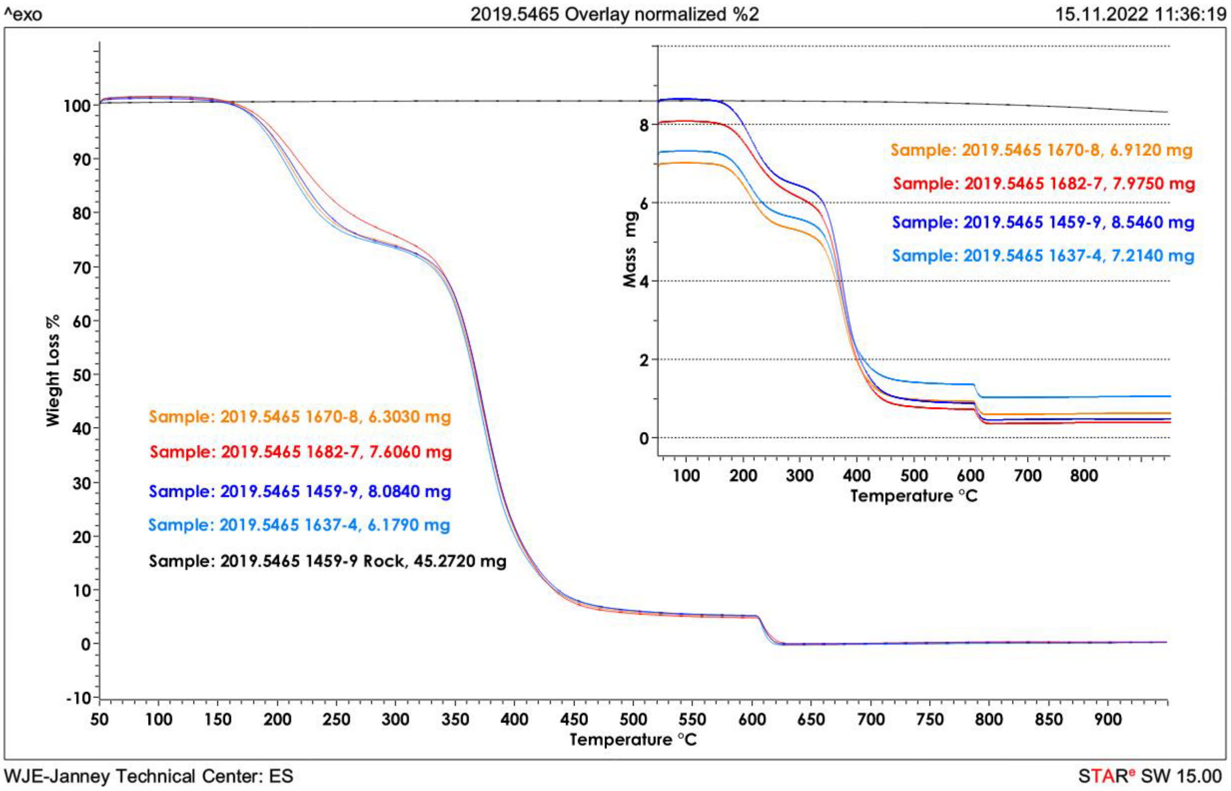
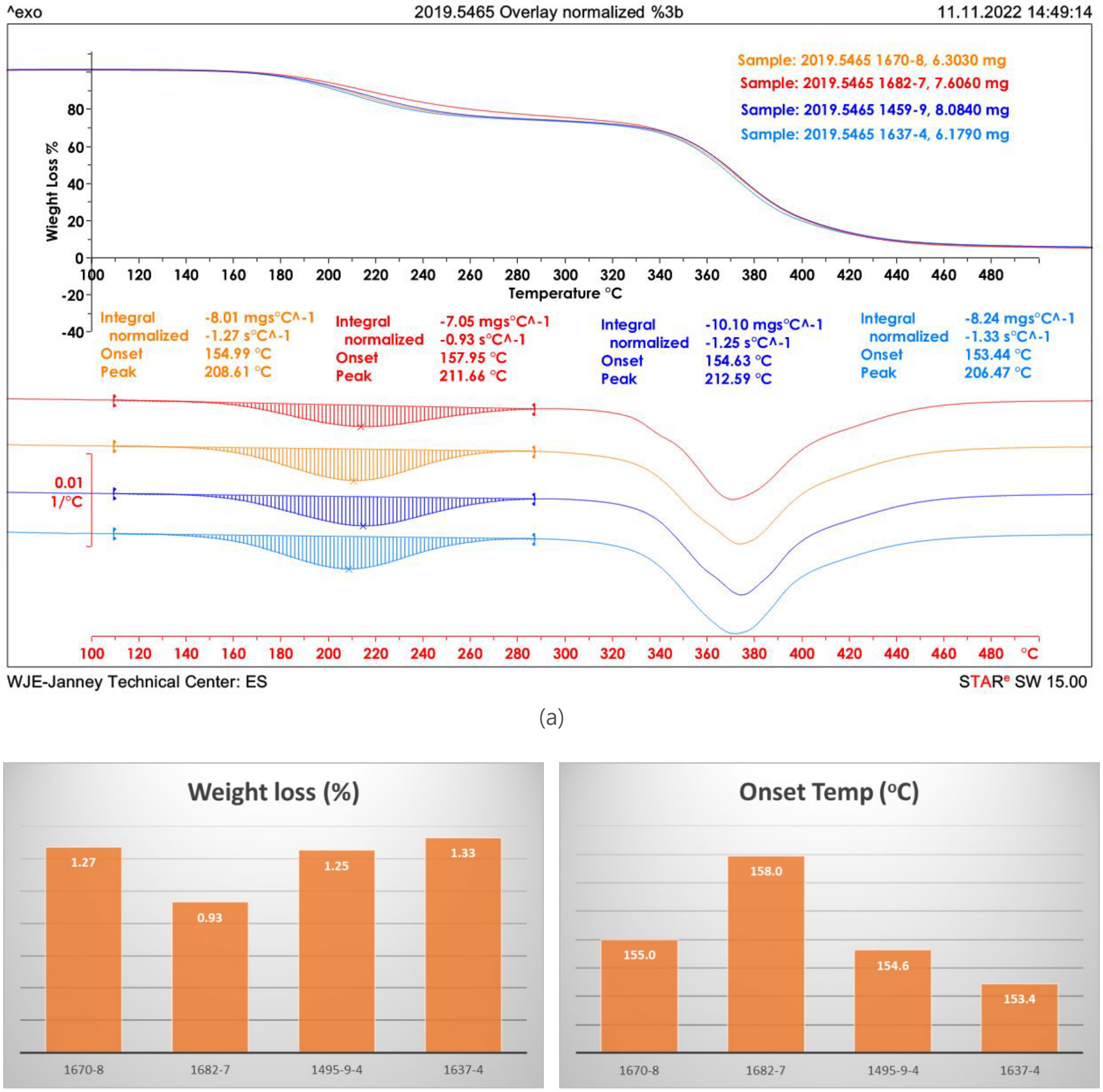


Figure 69. TGA thermograms of isolated binder samples: the atmosphere was switched from nitrogen to air at 600°C. The aggregates do not show any mass loss. As shown in the inset of mass change over temperature, noticeable amounts remain due to aggregate residues embedded in the isolated binding resins even after 950°C in air. The initial mass of the sample was calibrated using the remaining mass. The weight loss (%) of the binding resin alone was recalculated. The difference is observed below 350°C.

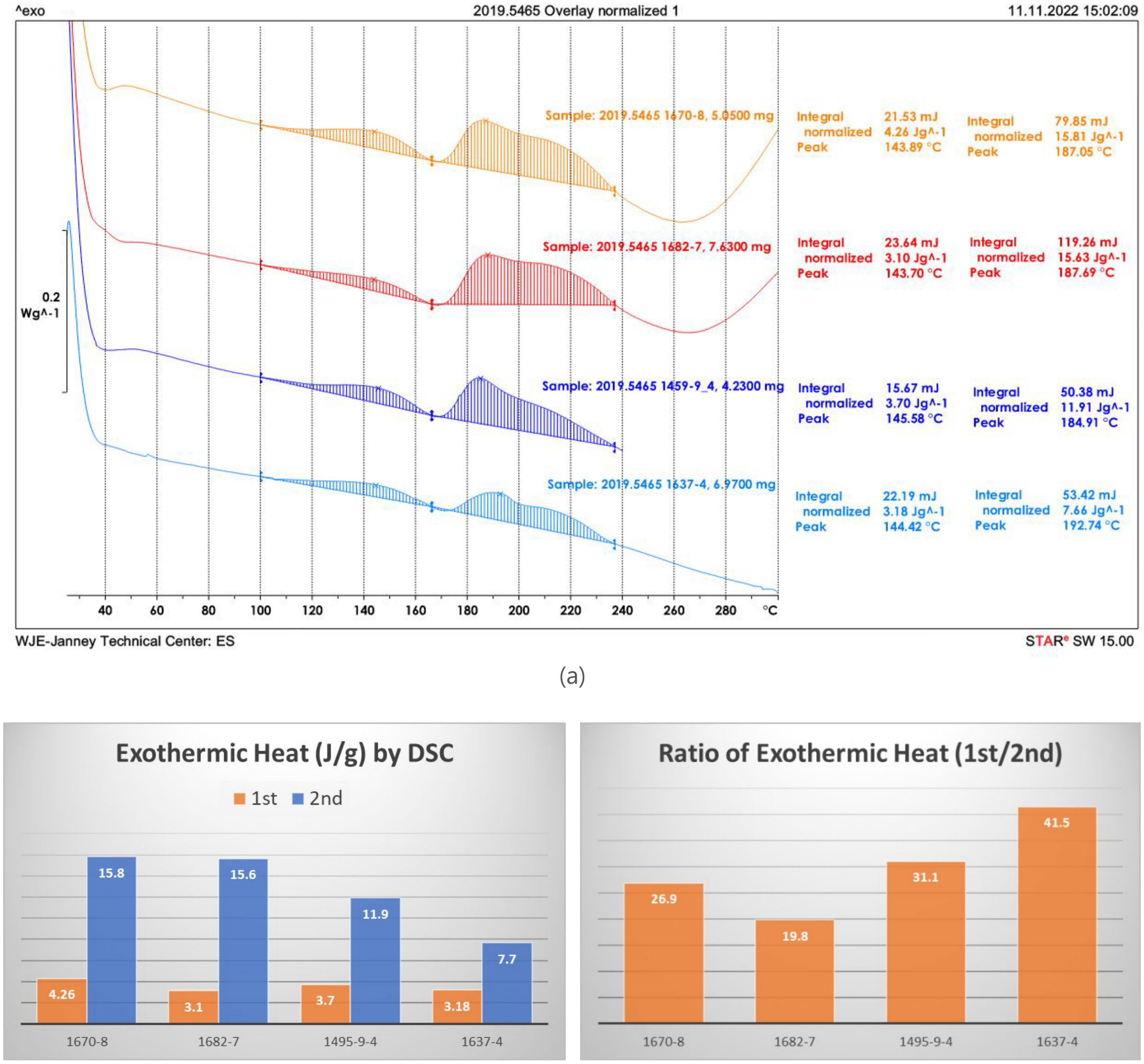


(a)

(b)

(c)

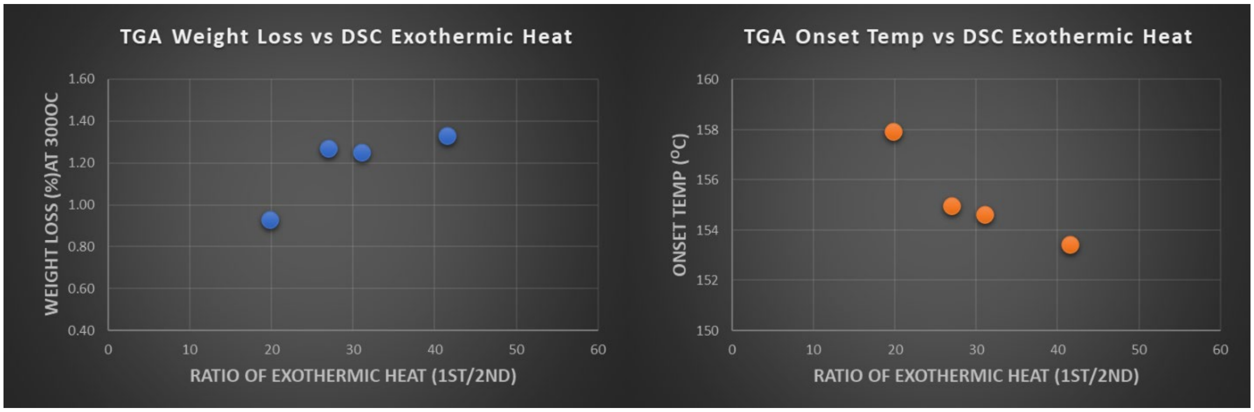
Figure 70. (a) TGA thermograms below 350°C (top) are further analyzed using DTG (1st derivative of thermogravimetry) curves (bottom) to measure (b) weight loss (%) and (c) onset temperature of weight loss. 1682-7 shows a smaller weight loss (%) and higher onset temperature indicating that the weight loss is delayed most.



(a)

(b) (c)

Figure 71. (a) DSC thermograms of isolated binder samples. Two exothermic peaks are observed: the lower peak is assumed to be a post curing portion of crosslinking that is not completed in service while the higher peak is the portion that remains intact. (b) Normalized exothermic heat amounts of the lower and the higher peaks by weight are measured but do not represent binding resins absolutely because the isolated samples include aggregate residues. (c) The exothermic heat ratio of 1st peak to 2nd peak is calculated. 1682-7 shows a smaller ratio, indicating that less post curing progressed.



(a) (b)

Figure 72. Positive correlation of weight loss (%) and negative correlation of onset temperature to the exothermic heat ratio: as the exothermic heat ratio of 1st peak to 2nd peak decreases and more curing occurs, the weight loss decreases and the onset temperature is delayed.

5.3.6.3. Summary of Results of Chemical Testing

When the HFST system is applied on a bridge deck, the low-viscosity polymer binder cures within several hours and hardens enough to achieve mechanical properties as well as bond strength. This is driven by a three-dimensional network formation of the polymer binder, the extent of which is related to the crosslinking density.

Based on the laboratory analysis conducted on several core binder samples, the epoxy-based polymer binders were observed to have different levels of crosslinking density. After the polymer binder reached the required crosslinking density for service, the crosslinking reaction continues to proceed slowly over time. The more connected the network structure is, the harder the polymer binder becomes. However, it brings about lower toughness and increased brittleness, which decrease binder elasticity.

The extent of crosslinking in a polymer resin is also influenced by environmental stresses such as ultraviolent light or ozone or physical aging by heat and is an expected phenomenon in polymer overlay systems. This testing indicated a small difference in the crosslinking density in one of the samples evaluated, which may have had an effect on aggregate retention, but this is judged to be a result of normal aging of the polymer resin as opposed to any serious deficiency in its fabrication or application.

6. DISCUSSION AND CONCLUSIONS

The performance and durability of HFSTs in Montana was assessed, particularly their ability to provide adequate skid resistance and protect Montana bridge decks from chloride-induced corrosion. Of particular interest was how varying traffic volumes and pre-existing deck conditions influence overlay performance and durability. Findings are based on a literature review and survey of select state transportation departments as well as site visits and laboratory analysis. Visual inspections and delamination surveys were conducted on fourteen Montana bridge decks across 2020 and 2022, and cores collected in 2020 and 2022 were tested to assess the HFST materials, macrotexture, chloride penetration resistance, and bond strength. Skid resistance testing of select HFSTs was also conducted in 2020 and 2023. The following discussion synthesizes the findings.

6.1. Discussion of Performance of Thin Polymer Overlays

Thin polymer overlays, HFSTs, and premixed polymer overlays can provide a skid-resistant surface while improving the resistance of the deck to moisture and chloride intrusion. The service lives of TPOs and HFSTs are expected to be about 5 to 15 years, but vary, and thicker PPC overlays are expected to have a longer life of 15 to 25 years and provide better protection to deicer ingress. Service life when placed on new decks or decks in good condition is expected to be longer than when placed on decks with active reinforcement corrosion or patched corrosion-related damage.

Cracking of the new wearing surface is not common unless it is reflective of crack or joint movement in the deck or at edges of deck patches related to patch shrinkage. Early-age distress can be due to many different construction-related mistakes (such as poor surface preparation or improper mixing) but long-term deterioration is usually limited to delamination and wear. Thin polymer overlays may have shorter service life when placed on heavily traveled roadways, especially with significant studded tire or chain abrasion wear.

Skid numbers tend to be high when epoxy-based thin polymer overlays are applied but can decrease in a year or two then stabilize somewhat. However, experience is widely variable. UDOT and PennDOT have prohibited the use of flint rock in HFSTs due to their tendency to polish and have poor long-term skid performance. NYSDOT noted that calcined bauxite tends to retain skid resistance well and basalt is also commonly used.

The thin HFSTs investigated in this study generally performed well. Adhesion and bond of properly formulated polymer overlays or HFSTs are excellent and cores tested from Montana exhibited bond strengths in excess of the deck concrete tensile strength. With respect to chloride penetration resistance, cored samples taken from the four bridge decks that underwent laboratory testing demonstrated excellent electrical resistance properties when tested in general accordance with ASTM C1202. In the field investigations, one overlay was observed to have numerous small, minor spalls after 7 years of service (the driving lane of Bridge 1670), but otherwise the overlays were generally intact and continuing to act as effective barriers to deicers.

6.1.1. Skid Resistance

With respect to skid resistance, typical forms of wear observed in the field investigation included aggregate fracture, aggregate pop-out, loss of aggregate agglomerations, and exposure of air voids entrapped in the polymer concrete matrix. Characteristic polishing and loss of surface aggregates were noted in wheel paths in the travel lanes after about two years of traffic wear. The amount of wear is generally related to traffic volumes based on visual inspections, laboratory testing, and skid resistance testing of the driving lanes, passing lanes, and shoulders of each bridge. Most HFSTs retained at least some roughness based on the average pavement macrotexture depths measured in the laboratory and the measured skid numbers. Skid resistance data was collected in 2020 for eight of the bridge decks studied and all of the overlays had average skid numbers of at least 50. A minimum skid number of 30 or 35 is commonly required, indicating that despite the wear observed, the HFSTs tested still retained sufficient skid resistance after up to 5 years of age. Skid resistance testing of select bridges included in this study in 2023 indicate that the skid numbers reduced by about 14% on average in 2023 compared to the corresponding value in 2020. The HFSTs still retained sufficient skid resistance with just one driving lane recording a skid number below 30 with the rest having a skid number above 39. The loss of skid resistance was more prevalent in the driving lane than the passing lane as seen in Figure 73 and Figure 74. Loss of skid resistance occurs within the first five years of service, but some decks still had adequate skid resistance after 8 years. No significant wear was noted in shoulder areas.

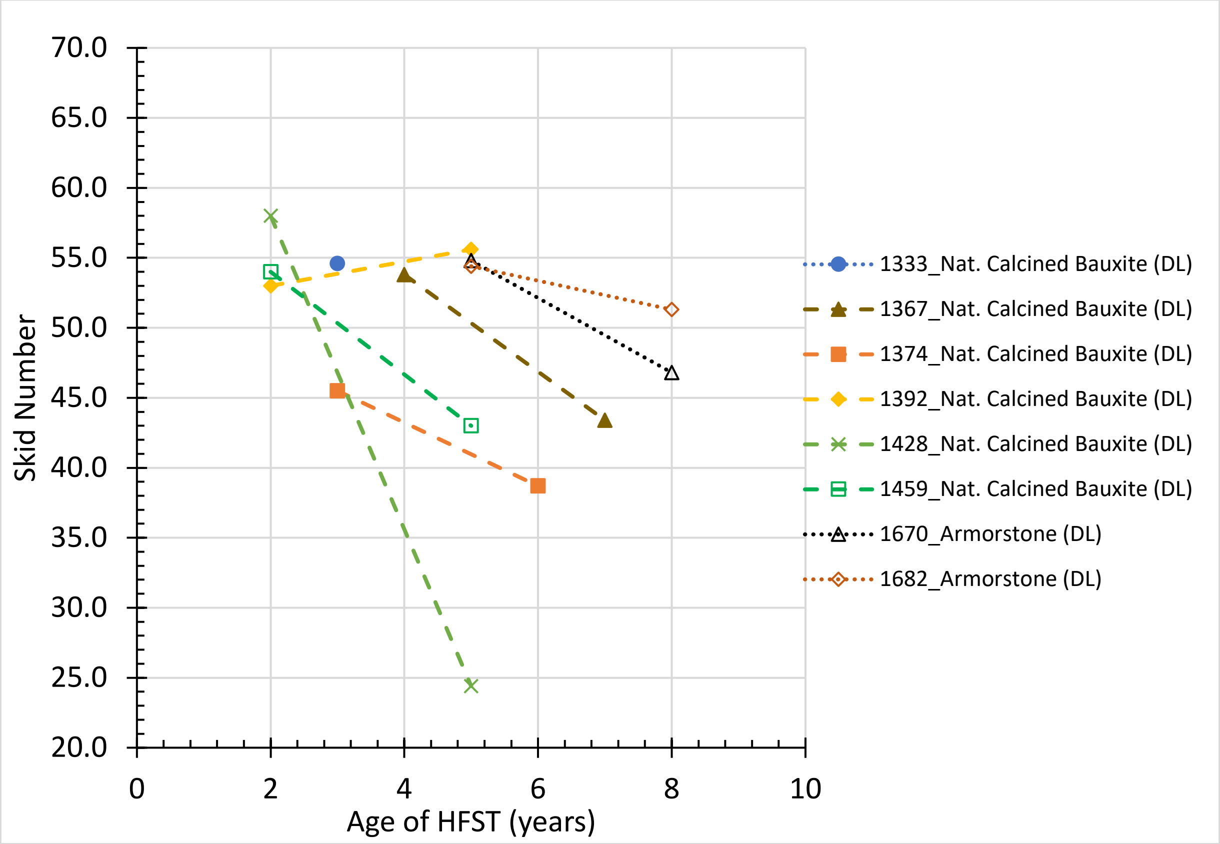


Figure 73. Comparison of driving lane (DL) skid numbers against age of overlay.

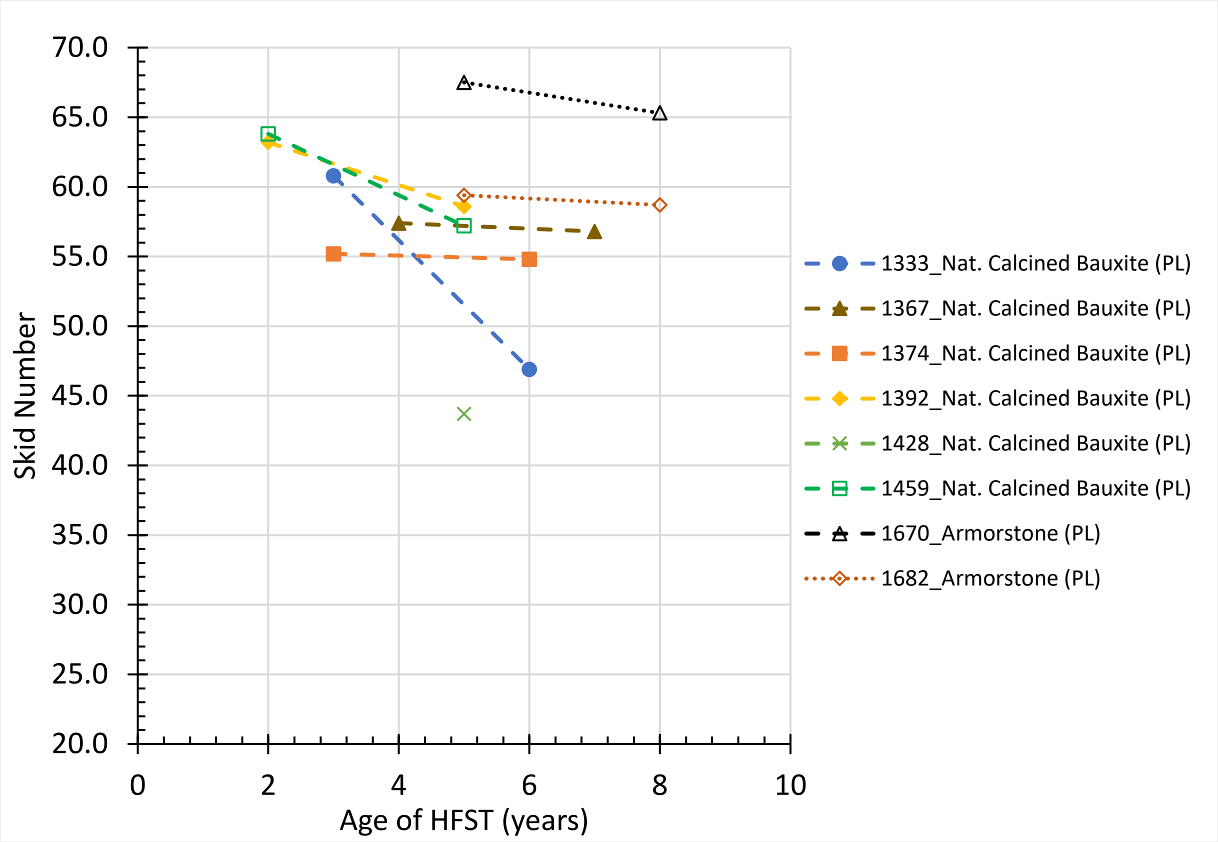


Figure 74. Comparison of passing lane (PL) skid numbers against age of overlay.

6.1.2. Quality of Overlay Materials and Installation

Overall, the results of the field and laboratory investigations indicate that the overlay materials and installations are generally of good quality for up to eight years of service. Evidence that the polymer experienced embrittlement at its top surface was noted with microscopic examination, which is expected as this surface is exposed to sunlight and oxygen. No visible signs of aging in the bulk of the overlays were noted, although material characterization indicated that the HFST on Bridge 1682 had a slightly greater degree of polymerization than the other three HFSTs that underwent detailed investigation. This was attributed primarily to 8 years of aging and not to any serious batching or installation issue. Microcracking was observed in some of the polymer matrices, particularly on Bridge 1367 during the 2020 lab testing and Bridge 1670 according to the 2022 field inspection. However, the microcracks appear to have a negligible impact on performance. The polymer did experience polishing under traffic, but the most recent skid testing results from 2023 indicate that skid resistance is still acceptable for most decks.

6.1.2.1. Aggregate

According to the petrographic examination, the aggregates used in the study bridges are basalt or calcined bauxite and are dense and durable. They did not polish under traffic, but instead tended to fracture near the binder surface. While fracture is not desirable, it is considered unavoidable and aggregate fracture is preferred over aggregate polishing as fractured surfaces can still provide a rough texture and aid in skid resistance.

6.1.2.2. Primer

The primary intent of the primer is to aid the bond between the overlay and the deck, but it can also penetrate and bond deck cracks. The impact of the primer on bond strength cannot be ascertained from the results of this investigation because all of the bond strength tests resulted in primary fractures in the deck substrate rather than at the overlay-deck interface. The bond strength test results therefore show that the overlays were well-bonded to the bridge decks with the use of the primer or not. However, primers are also beneficial because they were noted to penetrate existing cracks in the deck concrete, which helps provide protection in case the overlay wears away and may limit reflective cracking. The primer used on the study bridges in the Missoula District was capable of penetrating bridge deck cracks, as evidenced by Cores 1459-2 and 1367-3. Core 1459-2 had a visible crack that the primer penetrated to a depth of approximately 1.25 to 1.625 inches. Core 1367-3 had a hairline crack that the primer penetrated to a depth of 2 inches.

6.1.2.3. Bond Strength and Deck Delamination

The polymer topping bond strength remained good throughout the duration of this study and the topping bond strength exceeded the strength of the concrete substrate. Some deck delaminations were found and progressed over the study period but it was found that delaminations were due to continued corrosion of the top mat of reinforcing steel in the deck, not due to disbondment of the overlay. Small spalls in the HFSTs were found but most were only a few square inches in area and infrequent. Bridge 1670 was the exception and experienced numerous HFST spalls although they were mostly confined to the driving lane. The reason for the local spalling in the driving lane of Bridge 1670 may be due to a variety of reasons, including the omission of a primer, inconsistent surface preparation, or localized issues with curing. However, the small spalls do not significantly impact topping performance and it appears to be atypical compared to the performance of the other HFSTs investigated in this study.

Entrapped air voids or incomplete consolidation were observed across many of the HFSTs investigated in the laboratory and field inspections. Entrapped voiding is commonly due to an insufficient resin content to fill all voids. When the top surface aggregates were removed by traffic abrasion, the voids were exposed. The voids may help mitigate loss of skid resistance by providing macrotexture but may also reduce the moisture protection and life of the overlay.

6.1.2.4. Summary

In summary, the overlay systems were generally well-constructed such that a good bond strength and polymer concrete of good quality were achieved. Improved sealing of deck cracks is an advantage of using a primer. The materials used in the overlay systems similarly are of good quality, even though some material degradation, specifically embrittlement and microcracking of the top surface of the resin, aggregate fracture, and small areas of popouts have occurred. However, the overlays have still provided sufficient skid resistance and generally good protection against chloride penetration to date at ages of up to 5 to 8 years.

6.1.3. Influence of Climate on Overlay Performance

For the fourteen bridges investigated in Montana, the bridge location and regional climate do not appear to have impacted the performance of the overlays. A contour map of the average annual precipitation across Montana is shown in Figure 75 with the locations of the bridges investigated in this study identified on the map. Some regions of Montana receive only 6 to 12 inches of precipitation annually while other regions, primarily in the northwestern part of the state, receive over 85 inches of precipitation annually. Some of the bridges investigated are located in regions that receive as little as 12 to 14 inches of rain annually (near the city of Missoula) while others are located in regions receiving 22 to 34 inches of rain annually (near the city of St. Regis). Locations with more moisture would be expected to be more aggressive environments for HFSTs. However, the overlays consistently exhibited good performance.

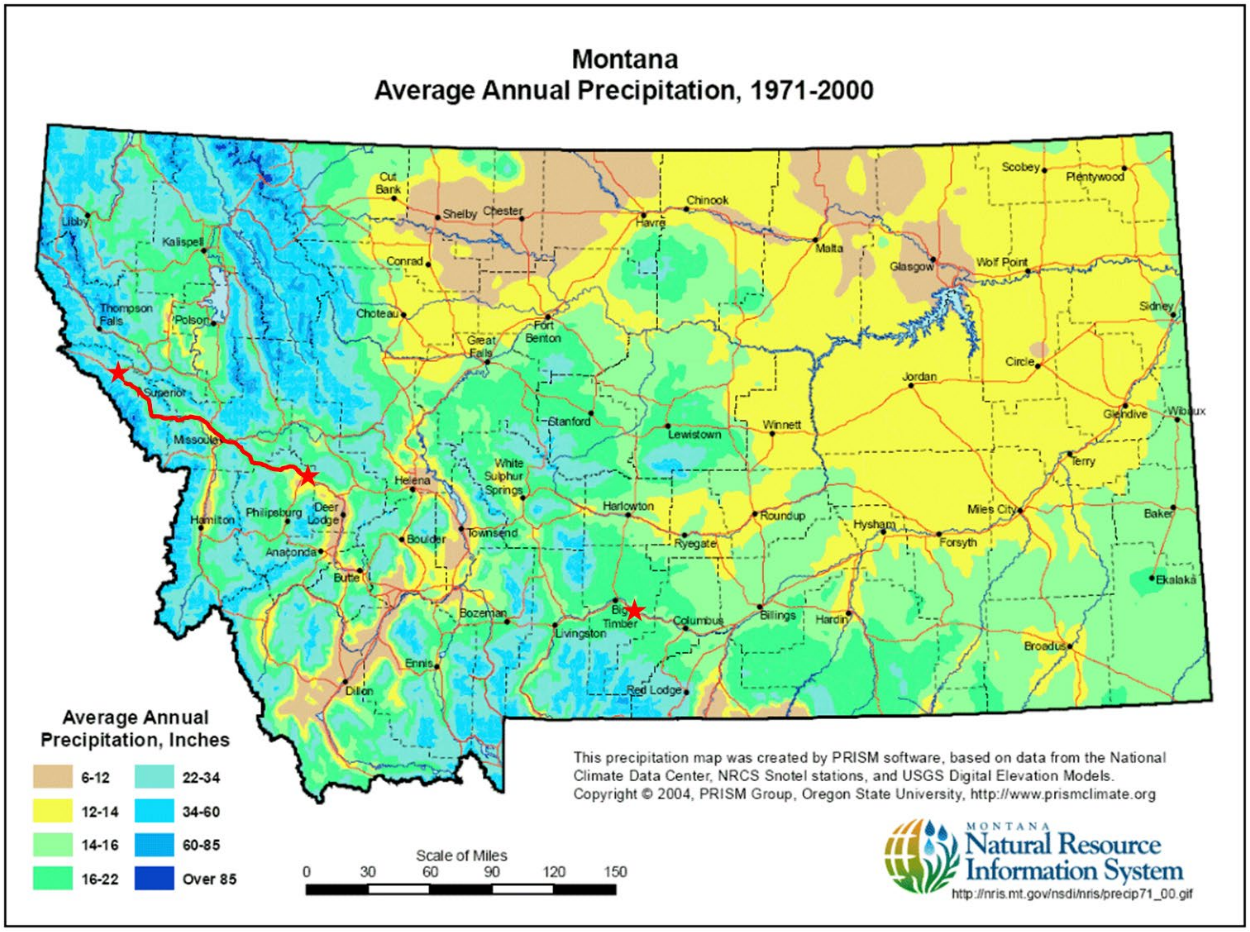


Figure 75. Map of average annual precipitation in Montana from Montana State Library (Montana State Library, 2022) with annotations by WJE. Red stars identify the location of the Billings bridges and the cities of Drummond and St. Regis, where the easternmost and westernmost bridges in the Missoula District are located. The remaining bridges in the Missoula District lay on the path between them, marked with a red line.

6.1.4. Influence of Traffic Volume on Overlay Performance

The performance of the HFSTs studied was compared to traffic volumes, especially truck traffic. In general, the HFST with the lowest traffic volume, Bridge 3734 with an ADT of 100 vehicles per day, had relatively "minor" wear based on the visual inspections compared to the other HFSTs in this study. Bridge 1428, which has the greatest ADT of 16,309 vehicles per day, had much greater wear with aggregate pop-outs and exposed voiding. Driving lanes exhibited more wear than passing lanes and shoulders did not typically show any signs of wear. However, wear did not correlate directly to ADT or ADTT. As an example, the HFST on Bridge 1392, which has an ADT of 9,138 vehicles per day, demonstrated "minor" wear after 4 years of service while the HFST on Bridges 14 and 25, which have a combined ADT of 15,747 vehicles per day, developed "typical" wear after only 2 years of service.

The skid numbers measured in both 2020 and 2023 show that increasing traffic or truck traffic volumes do not necessarily correlate to faster loss of skid resistance. Nine of the HFSTs investigated underwent skid resistance testing. The HFSTs were between 2 and 5 years of age in 2020, had ADT counts between 100 and 16,309 vehicles per day, and ADTT counts between 3 and 1,860 trucks per day. The 2020 skid numbers are plotted against the ADT in Figure 76 and against the ADTT in Figure 77. In 2023, the same HFSTs underwent skid testing again at ages of 5 to 8 years and their updated ADT counts as of 2023 were between 100 and 20,157 vehicles per day, with ADTT counts between 3 and 24,419 trucks per day. The 2023 skid numbers are plotted against the ADT and ADTT reported in 2023 in Figure 78 and Figure 79 respectively. The data is sorted based on the type of aggregate used and whether the data represents the driving lane (DL), passing lane (PL), or average (Avg). As noted previously, the passing lanes always had higher skid numbers than the driving lanes, which is due to the fact that greater traffic volumes cause increased wear. However, there is no clear correlation between skid number and bridge ADT or ADTT based on the data collected in this study.

Instead, the age of the HFST is a more significant factor. There is a negative correlation between the skid number and the age of the HFST, as shown in Figure 73 and Figure 74. Again, the data is sorted based on aggregate type and lane. The trend of decreasing skid resistance with increasing age holds true for the Missoula bridges tested, all of which have the calcined bauxite aggregate. In 2020, the 5-year-old HFSTs in the Billings District, which have the Armorstone (basalt) aggregate, had comparable performance to the 2-year-old HFSTs in the Missoula District. The amount of tire chain abrasion or studded tire exposure may be less on the Billing bridges than the mountain bridges in Missoula or other factors may be involved. Overall, both aggregate types performed adequately and direct comparisons cannot be done based on this limited study. Figure 80 and Figure 81 shows the plot of skid number against ADT again with the data categorized by HFST age instead of by aggregate type to show how the HFST age is more important than the ADT in terms of skid resistance.

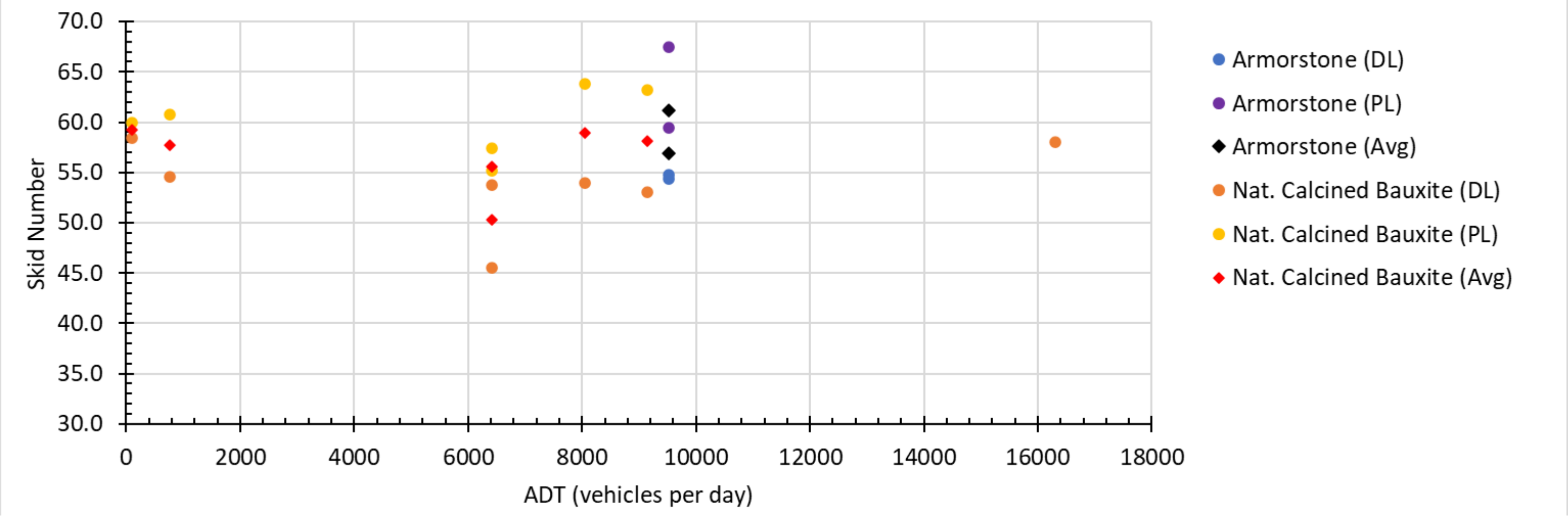


Figure 76. Plot of skid numbers measured in 2020 against ADT of bridge as reported in 2022. Data is categorized by aggregate source and lane type with “DL” representing the driving lane, “PL” representing the passing lane, and “Avg” representing the average value of the two lanes.

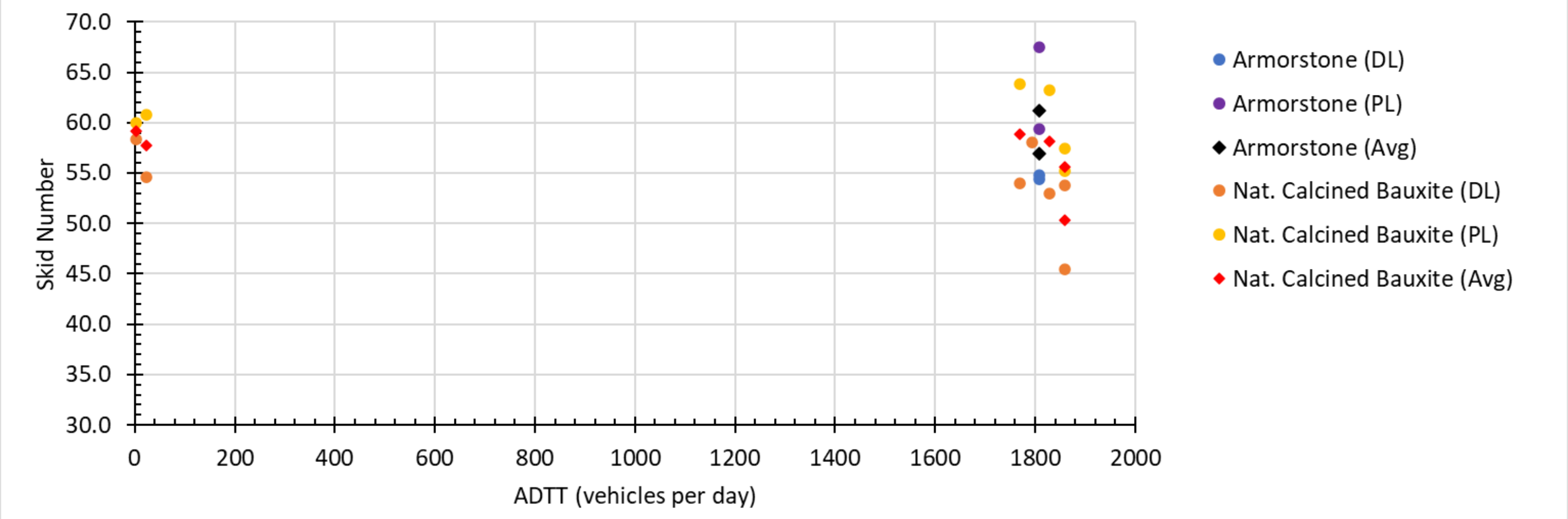


Figure 77. Plot of skid numbers measured in 2020 against ADTT of bridge as reported in 2022. Data is categorized by aggregate source and lane type with “DL” representing the driving lane, “PL” representing the passing lane, and “Avg” representing the average value of the two lanes.

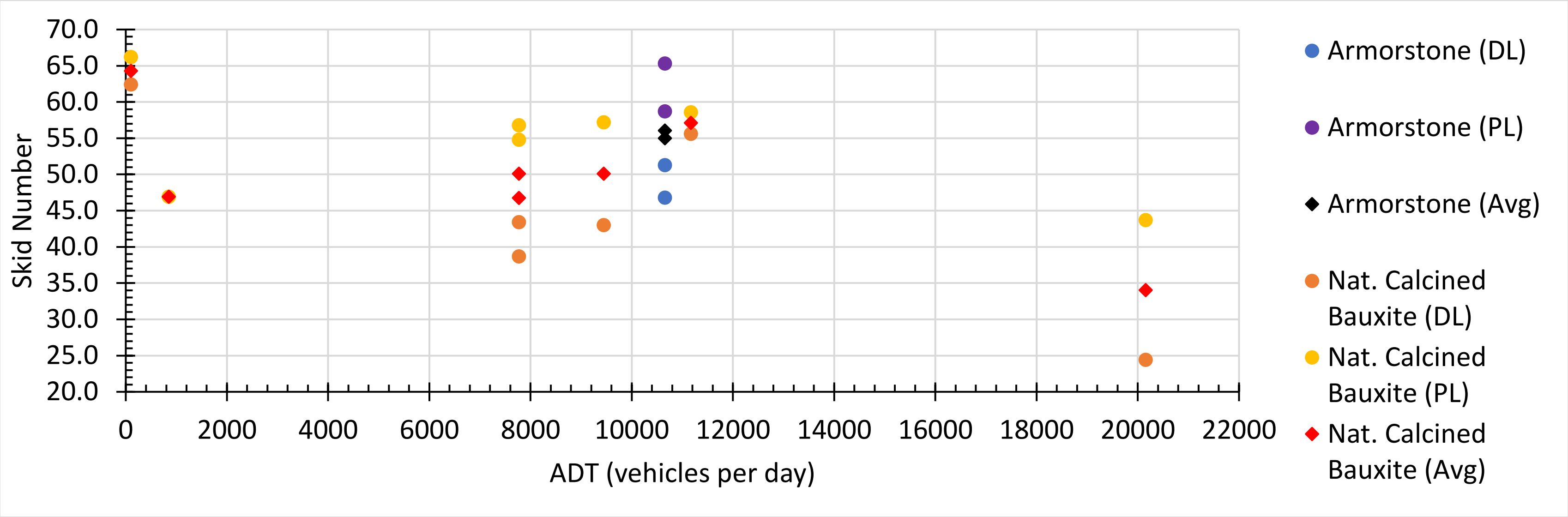


Figure 78. Plot of skid numbers measured in 2023 against ADT of bridge as reported in 2023. Data is categorized by aggregate source and lane type with “DL” representing the driving lane, “PL” representing the passing lane, and “Avg” representing the average value of the two lanes. Note that the vertical axis goes from 20 to 70 instead of 30 to 70 as for the plots of the 2020 data.

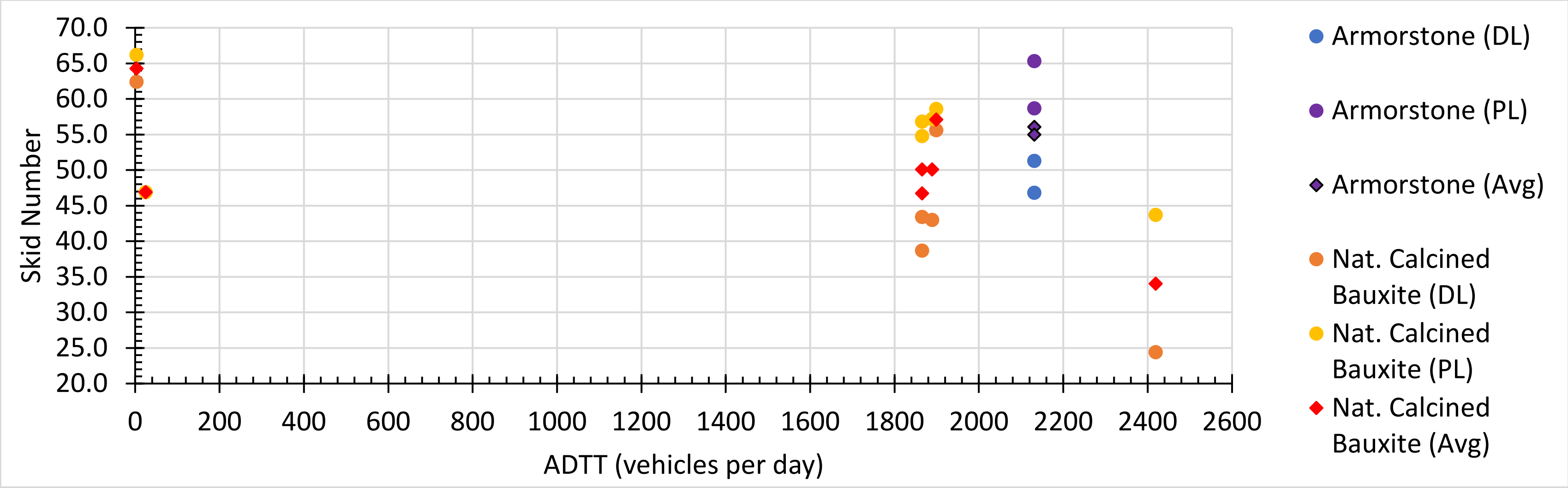


Figure 79. Plot of skid numbers measured in 2023 against ADTT of bridge as reported in 2023. Data is categorized by aggregate source and lane type with "DL" representing the driving lane, "PL" representing the passing lane, and "Avg" representing the average value of the two lanes. Note that the vertical axis goes from 20 to 70 instead of 30 to 70 as for the plots of the 2020 data.

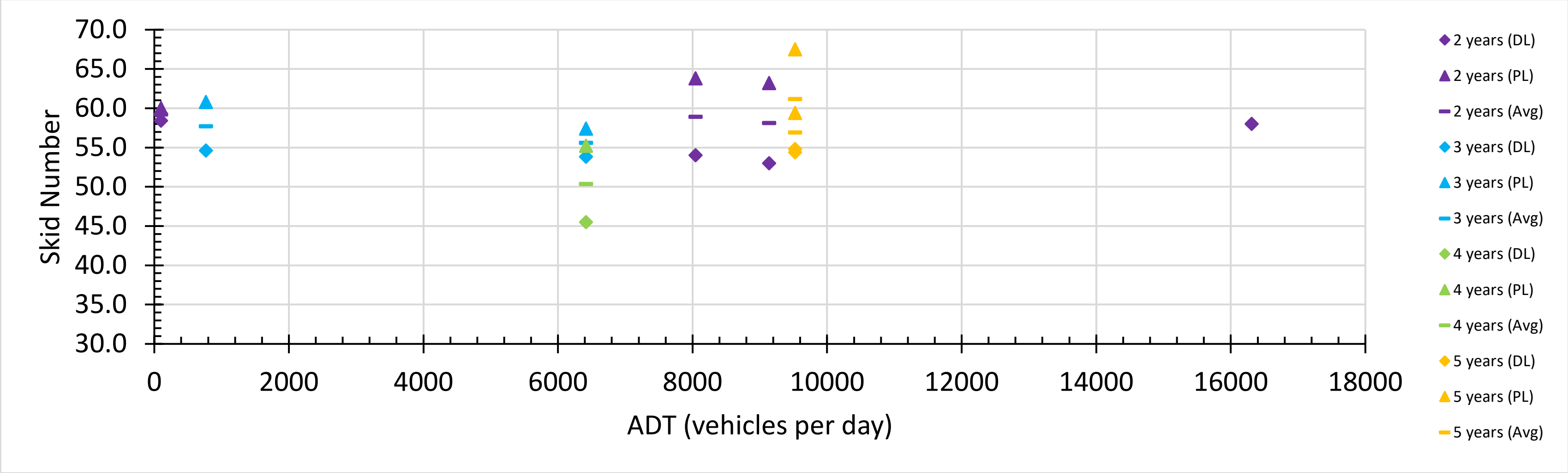


Figure 80. Plot of skid numbers measured in 2020 against ADT of bridge as reported in 2022. Data is categorized by age of the HFST and lane type with “DL” representing the driving lane, “PL” representing the passing lane, and “Avg” representing the average value of the two lanes.

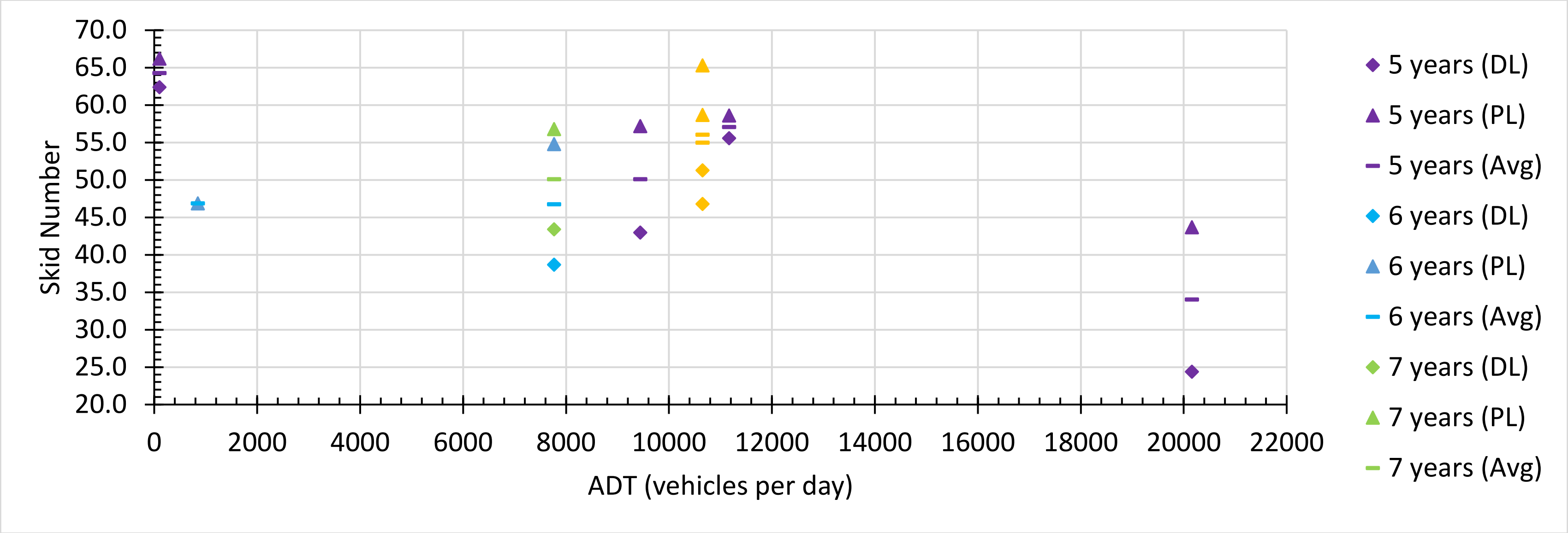


Figure 81. Plot of skid numbers measured in 2023 against ADT of bridge as reported in 2023. Data is categorized by age of the HFST and lane type with “DL” representing the driving lane, “PL” representing the passing lane, and “Avg” representing the average value of the two lanes. Note that the vertical axis goes from 20 to 70 instead of 30 to 70 as for the plots of the 2020 data.

6.1.5. Influence of Pre-Existing Condition on Overlay Performance

Many of the bridge decks had transverse cracks and required partial- and/or full-depth repairs when their HFSTs were installed. While the HFSTs had reflected transverse cracks to various degrees, the differences between the transverse crack maps of the HFSTs and those of the bridge soffits and the petrographic analysis of cores showed that many transverse cracks in the decks did not reflect in the thin HFSTs. While there were some instances of reflective transverse cracking, the thin HFSTs can help impede chloride penetration on a cracked bridge deck.

The presence of partial-depth repairs on bridge decks was of greater issue. The partial-depth repairs on the Billings bridges caused reflective cracking in the overlay at the perimeters of the repairs, and the patch perimeters had delaminations and spalls within 5 to 7 years of the placement of the HFST. The reflective cracking was likely due to the use of a polymeric patch material with relatively poor thermal compatibility with the deck or due to high shrinkage properties. Reflective cracking around repairs was not an issue in the Missoula bridges, which were given HFSTs after the Billings bridges and when the MDT had switched to more suitable cementitious patch materials and alternate polymer patch materials.

The presence of partial- and full-depth repairs shows that the bridge decks had already experienced initiation of chloride-induced corrosion by the time the thin HFSTs were applied. Delaminations under sound and well-bonded HFSTs were identified in 2022 as a result of continued reinforcement corrosion and progressive delamination. Polymer toppings are expected to limit moisture and chloride ingress and slow but not stop active reinforcement corrosion. The riding surfaces remain of good quality for now, but the delaminations found indicate that spalling and potholes should be expected in the future. The inspections in this study show that if corrosion is ongoing, delaminations will likely begin to be detected between 4 and 7 years after the installation of the HFST.

6.2. Conclusions

The conclusions of this study are as follows:

1. Northern climate Montana bridge decks are good candidates for polymer overlays or HFSTs. The epoxy HFSTs investigated in Montana have generally provided at least 5 to 8 years of satisfactory performance. New or lightly deicer-contaminated decks are likely to have HFSTs with less maintenance and a longer life than when HFSTs are applied to decks with active corrosion, delaminations, and significant chloride contamination. The inspections in this study show that if decks are chloride contaminated and corrosion is ongoing at the time of HFST installation, corrosion-related deck delaminations will likely begin to be detected between 4 and 7 years after the installation of the overlay.
2. Installation practices for the deck toppings investigated appear adequate. Overlay installations are generally of good quality, although optimization of multilayer application to reduce voiding within the polymer overlay matrix may be an improvement. The field studies were limited to epoxy-based thin HFST overlays and all overlays appeared to have adequate cure and hardness. The topping on Bridge 1682 was more fully cured and had a higher crosslinking density than Bridge 1367, but performance differences do not appear significant.
3. Deck repair prior to placing the HFST can affect performance. Reflective cracking at patch locations was likely due to the use of a polymeric patch material with relatively poor thermal compatibility with the deck or patch materials with high shrinkage properties. MDT has reportedly switched to more

suitable cementitious patch materials and alternate polymer patch materials that are more compatible with the deck and overlay and these patches appear to be durable and compatible with the overlay.

4. Sufficient bond of the HFST to the bridge decks was achieved and has been maintained. Bond strength tests fractured almost exclusively in the concrete deck substrate when conducted on deck overlay cores between 2 and 5 years of age.
5. The polymer toppings on the subject bridges had excellent electrical resistance and reduced chloride penetration into the deck up to 5 years of age. However, TPOs may slow but do not stop existing deck corrosion and corrosion-related delaminations were identified within 4 to 7 years of rehabilitation and overlay placement on actively corroding decks.
6. Transverse deck cracks reflected in the HFST of some decks and not others. Reflected cracking appeared to increase with time possibly due to thermal cycling, fatigue, or resin embrittlement. The primer used on the select bridges was able to penetrate into existing deck cracks and may help prevent reflective cracking. Cracking of the HFST should be expected over continuous piers where deck cracks are active.
7. Wear of the HFSTs on the decks studied occurred at wheel paths within the first two years and resulted in fracturing of the surface aggregate to the level of the resin embedment and loss of aggregate and aggregate agglomerations resulting in areas of pop-outs. The adhesion of the resin to the deck and between the resin and aggregate remained good. Surface (resin) polishing and microcracking and embrittlement of the resin surface was noted by microscopic examination but has not appeared to adversely affect performance to date.
8. The thin HFSTs exhibited good skid resistance up to 5 years of service and some decks up to 8 years of service. Wheel paths typically exhibit wear within the first two years with the driving lane consistently showing more wear than the passing lane. For some bridges skid numbers were less than 30 or 40 after five years but others have maintained good values through 8 years (Bridges 1670 and 1682 in the Billings District and with Armorstone (basalt) aggregates). The age of the overlay has a greater impact on its skid resistance than the traffic or truck traffic volume. The Missoula bridges tested, all of which have the calcined bauxite aggregate, showed decreases in skid resistance after 3 years. The HFSTs in the Billings District, which have the Armorstone (basalt) aggregate maintained skid resistance up to the eight years and had comparable skid numbers to 2 to 5-year-old HFSTs in the Missoula District. Note that the amount of tire chain abrasion or studded tire exposure may be less on the Billing bridges than the mountain bridges in Missoula or other factors may be involved.
9. Snowplow impacts and traffic caused wear and loss of the polymer topping at local areas typically along the approach joint and at occasional gouges. Elevation differences and uneven joint conditions can affect the impact and wear of the overlay.
10. No limitations on the geographic use of the polymer HFSTs were identified or are expected in Montana.

7. RECOMMENDATIONS

The recommendations to the MDT based on the findings of this study are:

1. The conclusions and recommendations of this report only apply to the polymer HFST or overlays investigated in this study. Other polymer formulations are likely to perform differently and should be evaluated separately. Trial installations and evaluation are recommended unless standard materials are being used and the contractor is well experienced.
2. Use only cementitious repair materials for deck patching that are compatible with the polymer topping or other rapid setting materials shown to be compatible and having acceptable performance when used prior to placing polymer HFSTs. Avoid patch materials that are thermally incompatible or have high shrinkage. The polymer topping adhesion to any new patch material should be tested prior to use. Have contractors map locations and specifics of deck repairs prior to placing toppings and keep in project files.
3. Address issues with incomplete consolidation and entrapped air voids within the HFSTs by requiring that the contractor demonstrate that the resin content is appropriate in a trial demonstration, or through evaluation of the in-place overlay. Back rolling the first layer or an optimization study may be valuable.
4. Improve detailing at the bridge approach joint. Control and match elevations across the joint. Extend the thin HFST some distance, e.g., approximately 10 feet, beyond the bridge ends if the approaches are portland cement concrete to minimize vertical offsets and reduce snow plow damage and edge wear of the overlay on the bridge deck. Consider grinding existing deck along the approach joints to increase thickness of polymer topping along this edge.
5. Continue to monitor skid resistance of the HFSTs. Data pertaining to driving lanes and passing lanes should be kept in separate datasets instead of averaged. Additionally, the data should be categorized by aggregate source (type) in order to develop appropriate expectations for the performance of the various aggregate types and HFST systems and their appropriateness across different exposures.
6. Armorstone (basalt) appears to maintain skid resistance longer than naturally occurring calcined bauxite aggregate; however, differences in deck exposures of the study bridges may affect performance. A Mohs hardness of at least 7 is preferred and some states prohibit the use of flint rock in HFSTs due to their tendency to polish and have poor long-term skid performance.
7. HFSTs in this study lost surface friction before wearing through. Ideal HFSTs would maintain surface friction throughout their life. Surface friction and wear rely on the aggregate properties as well as the polymer resin modulus and toughness. New resin formulations that do not polish and maintain skid resistance as they wear is a focus for research.
8. Evaluate and test if an additional layer of HFST may be applied on top of the existing overlay. Consider reapplication of HFST to driving lanes after five years to restore skid resistance and extend deck protection.
9. Favor new bridge decks and decks without signs of corrosion initiation as candidates for thin polymer overlays or HFSTs. However, bridge decks in need of local full- or partial-depth patches do not need to be precluded from consideration. Corrosion testing, half-cell potential surveys, and determination of chloride contents in the deck can aid in optimizing deck selection. Avoid use on decks with widespread damage due to reinforcing corrosion (decks near the end of their service life).

10. Transverse deck cracks tend to reflect with time. Primer was noted to penetrate deck cracks and may help reduce reflective cracking.
11. While current practice appears adequate, achieving good bond is critical to polymer overlay performance. Implement quality assurance/quality control testing to ensure adequate surface preparation (concrete surface profile (CSP) of at least 5) and to monitor polymer batching, mixing, placement, and curing. Depending on the deck surface condition, micromilling may be advantageous to remove surface contamination and chloride-contaminated concrete.
12. Consider a 5-year warranty clause as specified by other states.

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APPENDIX A. SURVEY RESPONSES

The raw survey responses as downloaded from the survey tool (SoGoSurvey) are presented in the order listed in Table A.1. Appendices are available on the project web page:
https://www.mdt.mt.gov/research/projects/const/evaluation.aspx

Table A.1. Order of Survey Responses

	Responding Agency's Name (Shortened Name)
1.	Alberta Ministry of Transportation (Alberta Transportation)
2.	California Department of Transportation (Caltrans)
3.	Colorado Department of Transportation (CDOT)
4.	Michigan Department of Transportation (MDOT)
5.	New York State Department of Transportation (NYSDOT)
6.	North Carolina Department of Transportation (NCDOT)
7.	North Dakota Department of Transportation (ND DOT)
8.	Oregon Department of Transportation (OregonDOT)
9.	Pennsylvania Department of Transportation (PennDOT)
10.	South Dakota Department of Transportation (SDDOT)
11.	Utah Department of Transportation (UDOT)
12.	Washington Department of Transportation (WSDOT)

APPENDIX B. SPECIFICATIONS AND APPROVED PRODUCTS LISTS

Appendices are available on the project web page: https://www.mdt.mt.gov/research/projects/const/evaluation.aspx. The approved products lists, standard specifications, and special provisions identified by the survey respondents are compiled in this appendix. The documents or document sections are organized by agency:

Alberta Transportation:

- Standard Specifications for Bridge Construction, Section 15 Non-Skid Polymer Overlay
- B405-July 00: Specification for Polymer Resins Used in Polymer Overlays
- B392 – July 2000: Specification for Seed Aggregates Used in Polymer Membranes and Overlays

Caltrans:

- Standard Specifications, Section 60-3.04 Deck Overlays
    - 4-19-2019 Revision: Replace the 9th paragraph of Section 60-3.04B(3)(c) with: *Protect the overlay from moisture and do not allow traffic or equipment on the overlay (1) for a minimum of 4 hours cure time after final finishing and (2) until each rebound test result for the final finish shows a reading of at least 28 when tested under ASTM C805. The cure time must be extended if ordered. The rebound test may not be used to reduce the 4-hour cure time of the overlay.*

MDOT:

- Special Provision for Thin Epoxy Polymer Bridge Deck Overlay
    - Contains approved products list for two-component 100 percent solids epoxy systems
    - Contains list of approved aggregate suppliers
- Special Provision for High Friction Surface Treatment

NYSDOT:

- Technical Services – Materials – Approved List, Thin Overlays, Structural
    - Thin Polymer (Epoxy) Overlay Wearing Surface for Structural Slabs (584.50010018)
    - Approved Aggregates for Use with (584.50010018)
    - High Friction Aggregate
- Item 584.40000009 – Polymer Overlay Wearing Surface for Structural Slabs (PPC)
- Item 584.50010018 – Thin Polymer (Epoxy) Overlays for Structural Slabs
- Item 601.03000004 – Specialty Friction Surface Treatment for Concrete

NCDOT:

- Epoxy Overlay System I
- Epoxy Overlay System II

OregonDOT:

- Oregon Standard Specifications for Construction
    - Section 00556 – Multi-Layer Polymer Concrete Overlay
    - Section 00557 – Premixed Polymer Concrete Overlays

SDDOT:

- Standard specifications for Roads and Bridges
    - Section 491 Bridge Deck Polymer Chip Seal
    - Section 805 Materials for Polymer Chip Seals

UDOT:

- 2020 Standard Specifications for Road and Bridge Construction, Section 03372 Thin Bonded Polymer Overlay

APPENDIX C. MATERIAL SUBMITTALS AND PRODUCT DATASHEETS FOR HFSTS USED BY MDT

This appendix contains the following: (1) the special provisions related to deck repair and overlay operations for Contract No. JCC16, under which many of the Missoula District bridges were overlaid; (2) the project submittals from L & J Construction Group, LLC for Project JOC STPB STWD (477) in which many of the Missoula District bridges were overlaid; and (3) the submitted datasheet and supporting info for the Armorstone aggregates used on the bridges in the Billings District. The project submittal for the Missoula District bridge overlays includes the technical datasheets, material safety datasheets, and supporting laboratory test reports for all of the products used in the project, i.e., the primer Pro Poxy 45, the resin binder for the overlays Pro-Poxy Type III DOT, the epoxy repair mortar kit Sure Patch, the calcined bauxite aggregates from the Lake Ranch pit, and the expansion joint sealant MasterSeal SL 1. Appendices are available on the project web page: https://www.mdt.mt.gov/research/projects/const/evaluation.aspx

APPENDIX D. FIELD NOTES FROM 2020 AND 2022 INSPECTIONS

Appendices are available on the project web page: https://www.mdt.mt.gov/research/projects/const/evaluation.aspx

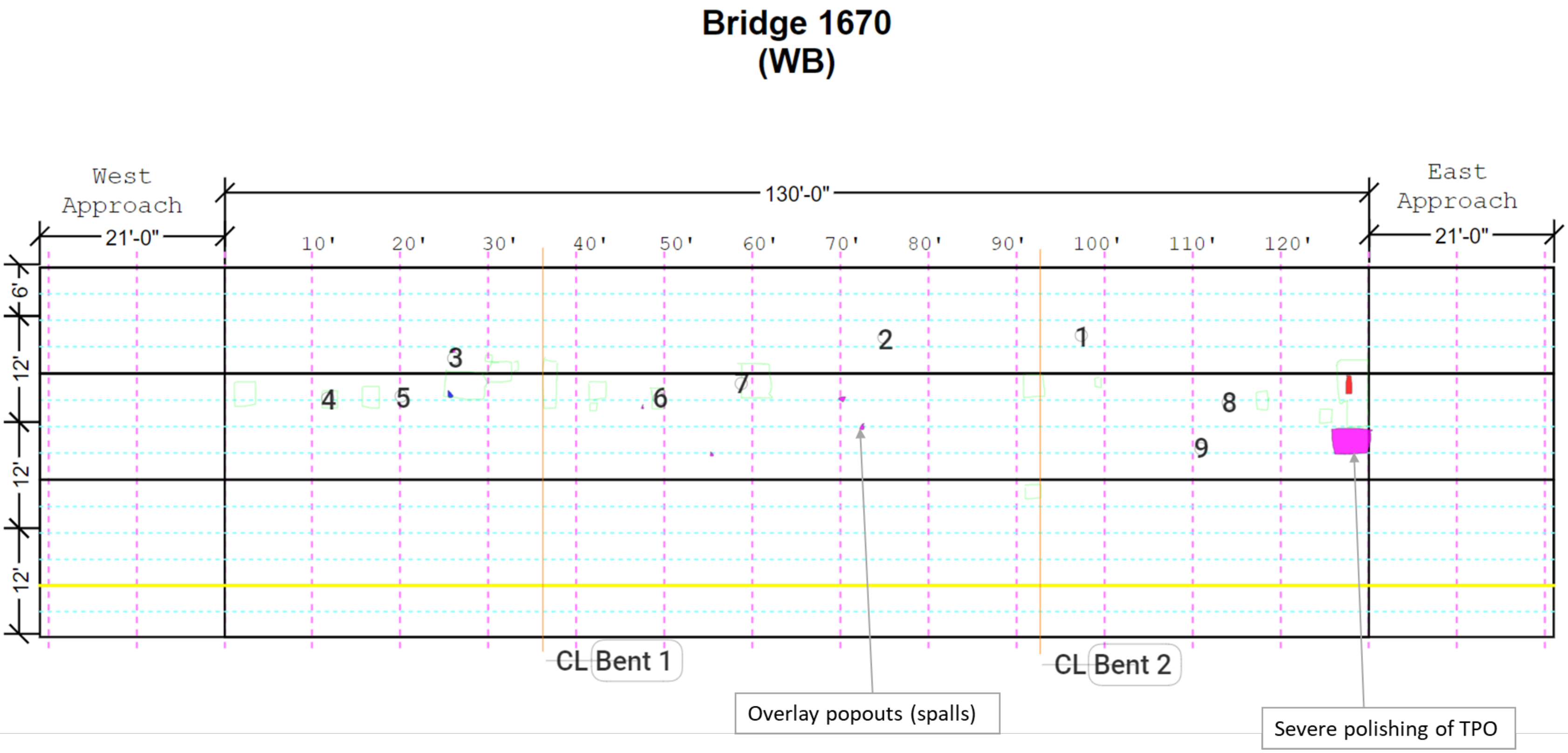

Figure D.1. Field notes for Bridge 1670, 2020 inspection, HFST.

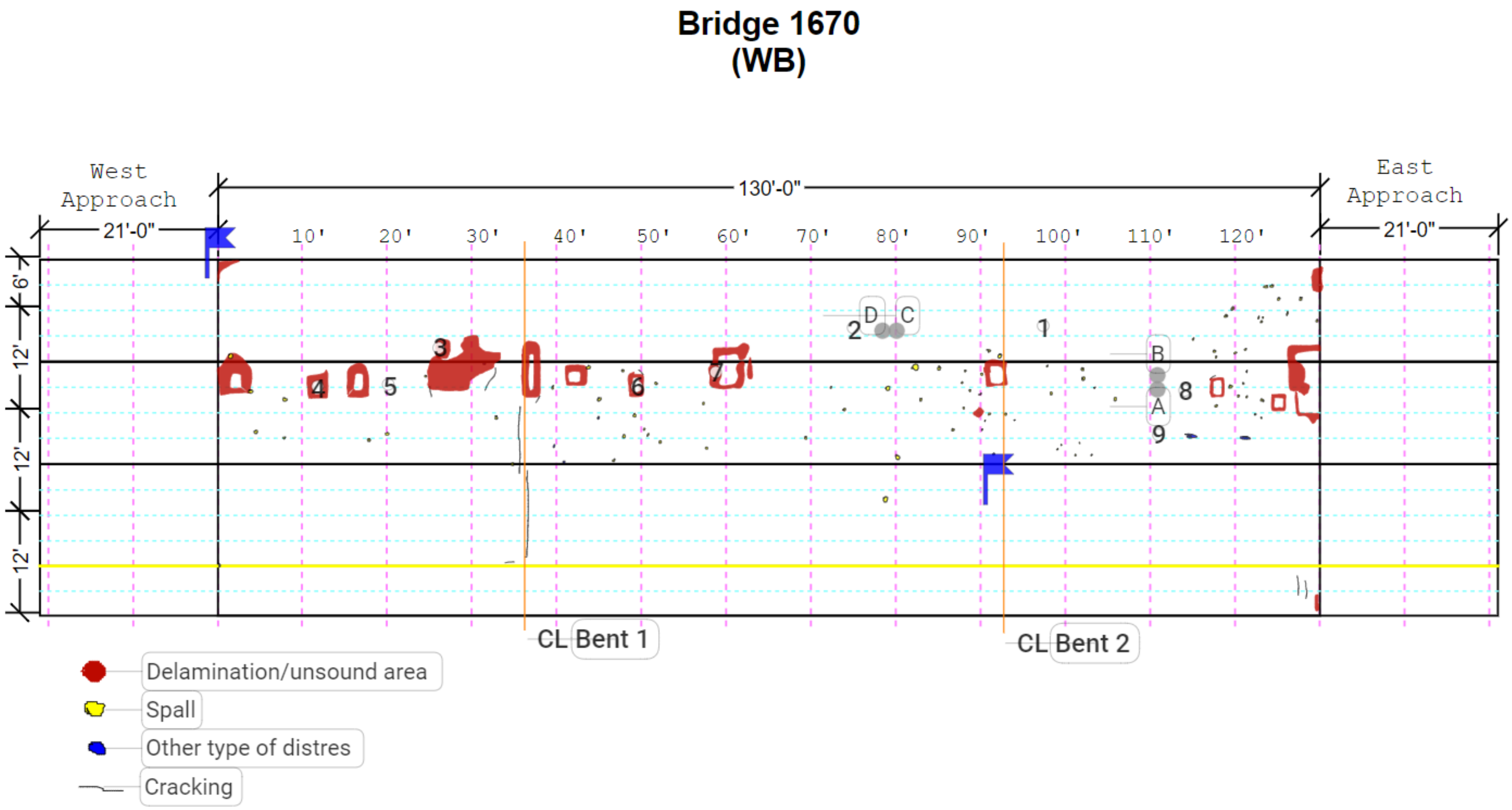

Figure D.2. Field notes for Bridge 1670, 2022 inspection, HFST.

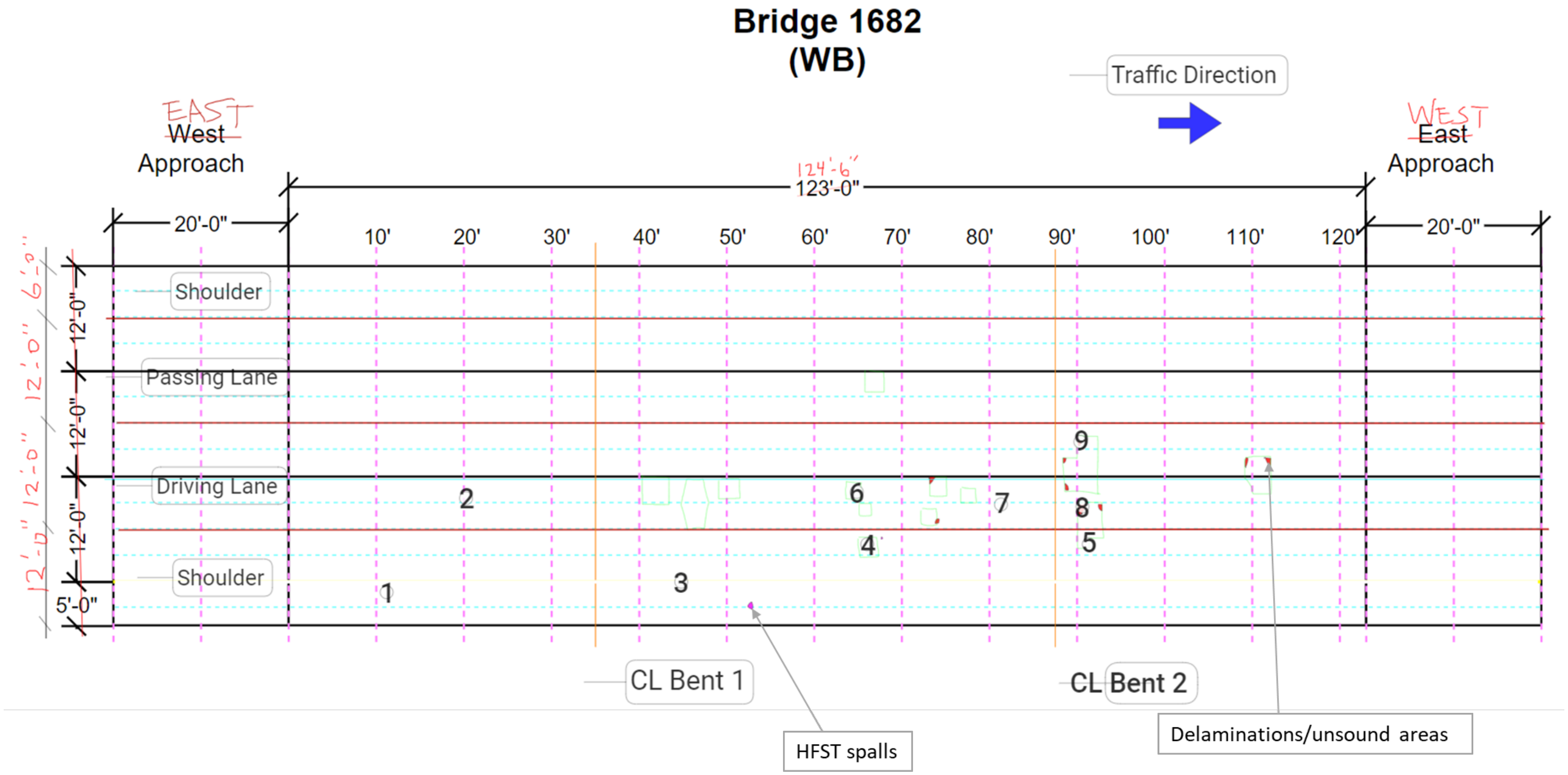

Figure D.3. Field notes for Bridge 1682, 2020 inspection, HFST.

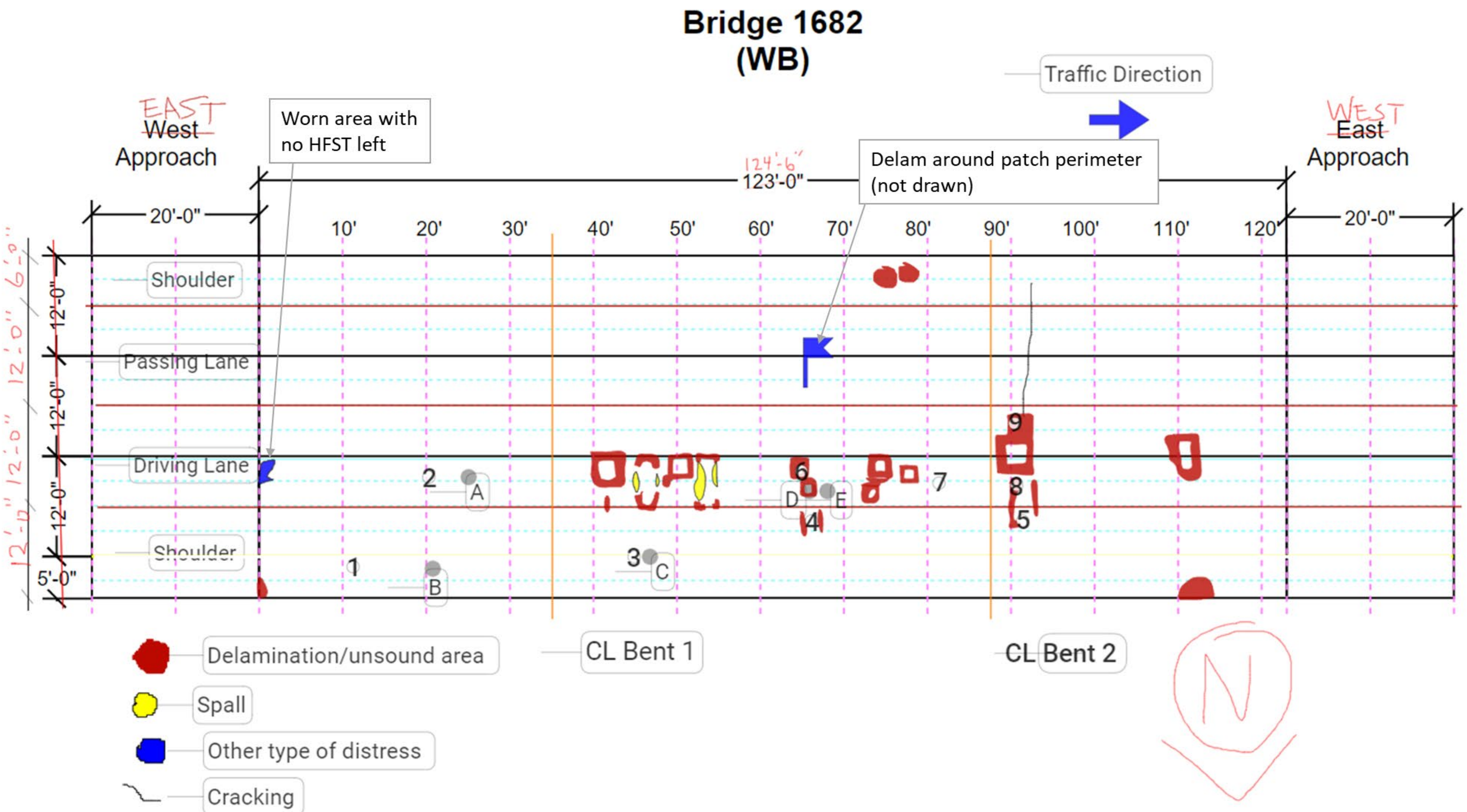

Figure D.4. Field notes for Bridge 1682, 2022 inspection, HFST.

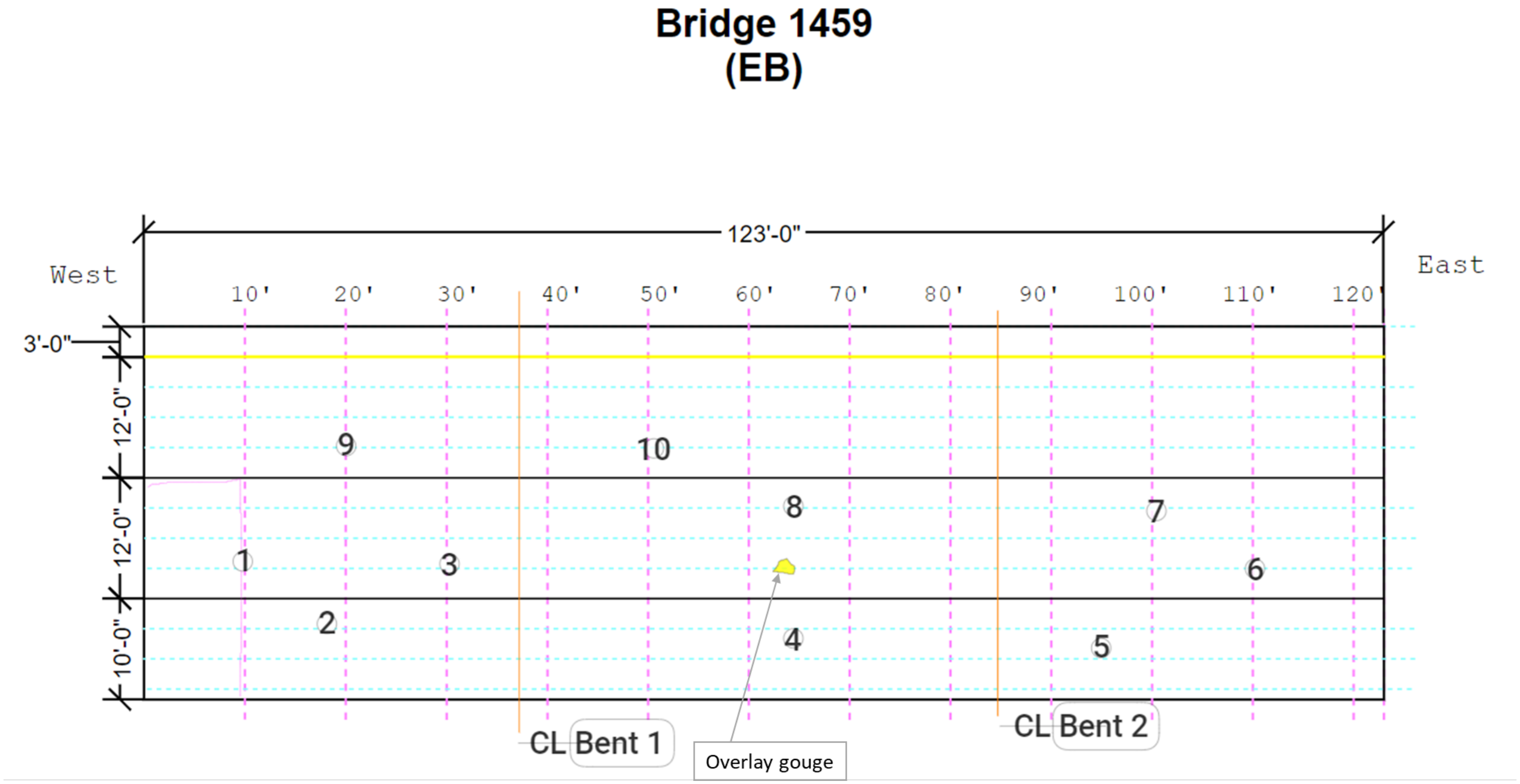

Figure D.5. Field notes for Bridge 1459, 2020 inspection, HFST.

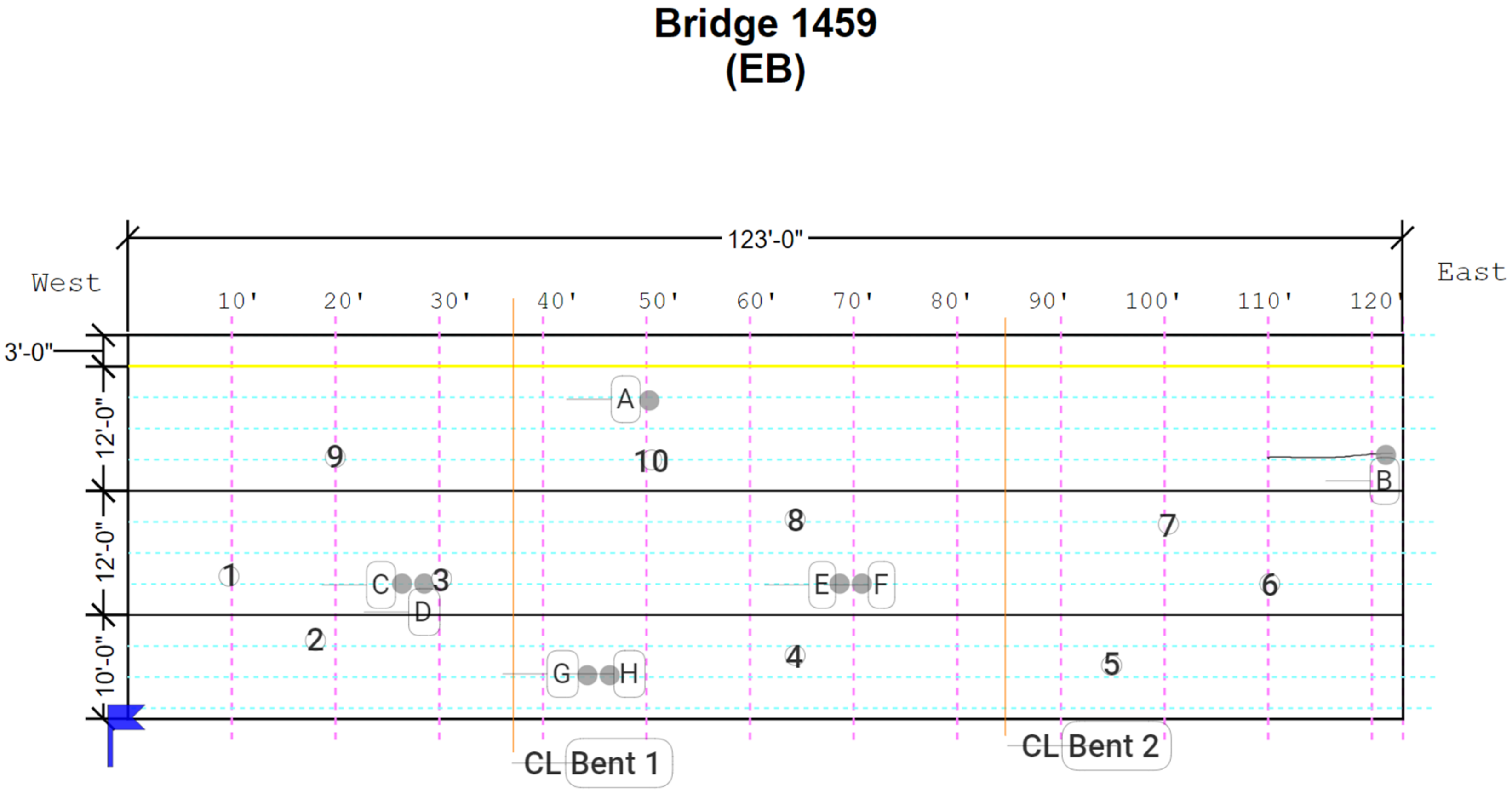

Figure D.6. Field notes for Bridge 1459, 2022 inspection, HFST.

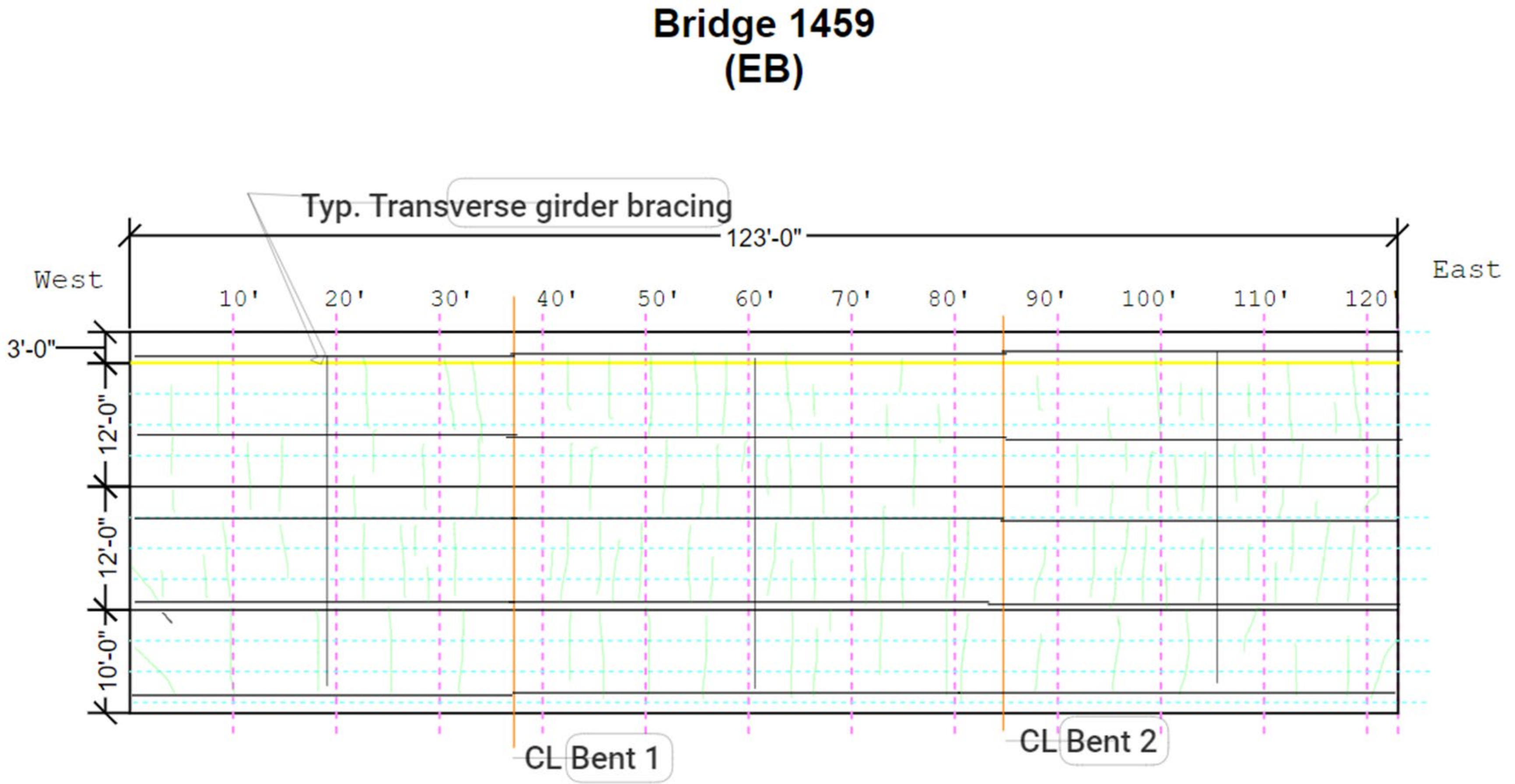

Figure D.7. Field notes for Bridge 1459, 2020 inspection, soffit.

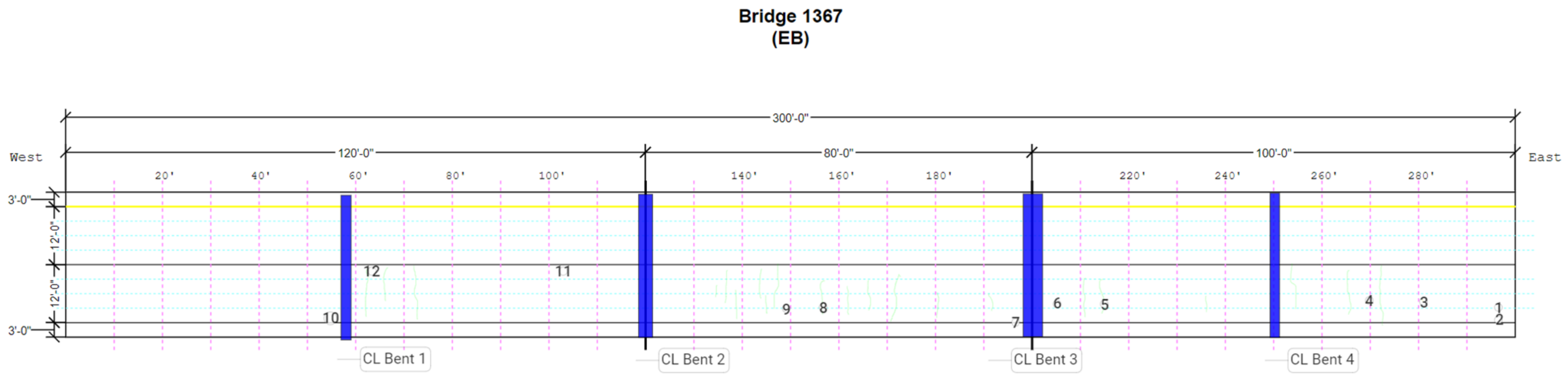

Figure D.8. Field notes for Bridge 1367, 2020 inspection, HFST.

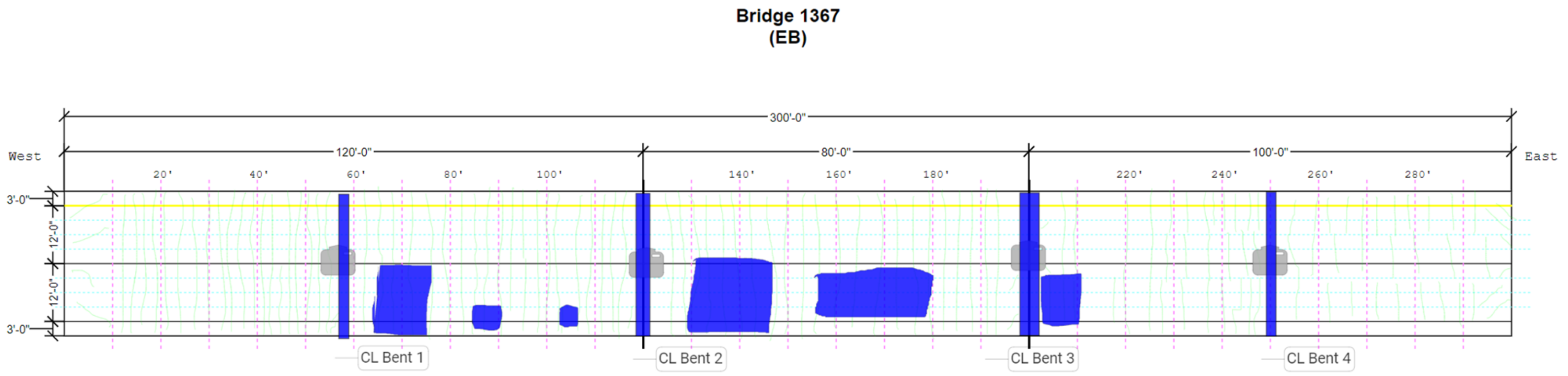

Figure D.9. Field notes for Bridge 1367, 2020 inspection, soffit.

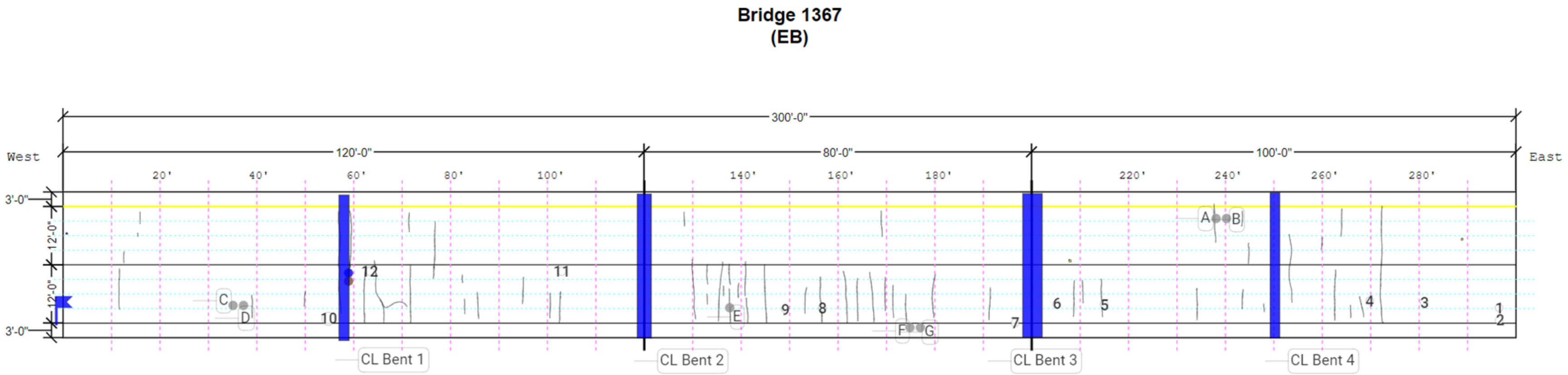

Figure D.10. Field notes for Bridge 1367, 2022 inspection., HFST.

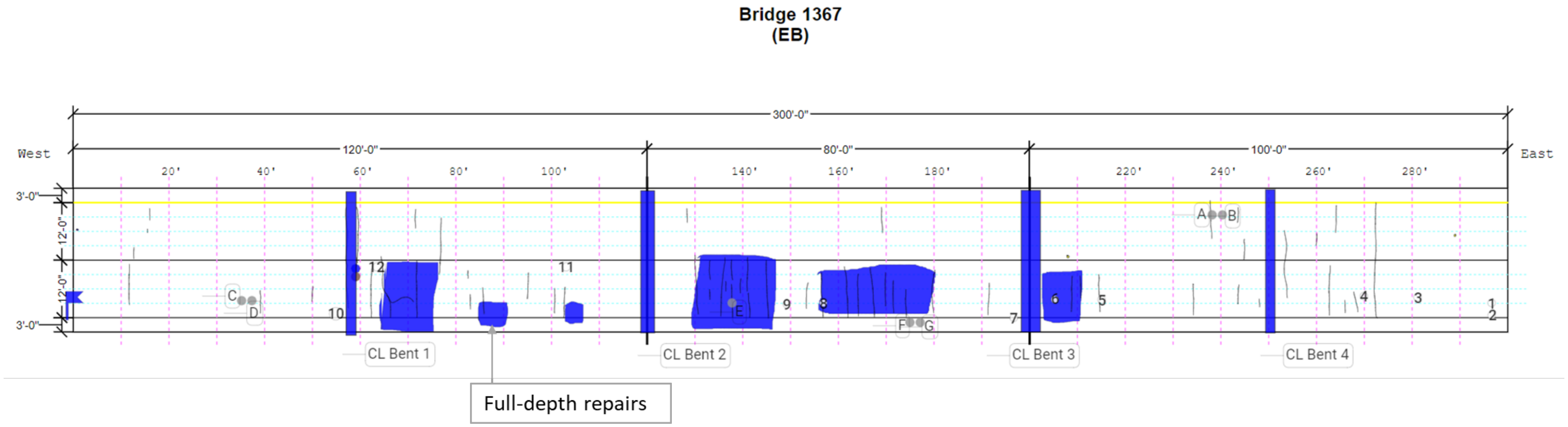

Figure D.11. Field notes for Bridge 1367, 2022 inspection, HFST, with full-depth repairs as identified from 2020 soffit inspection for comparison.

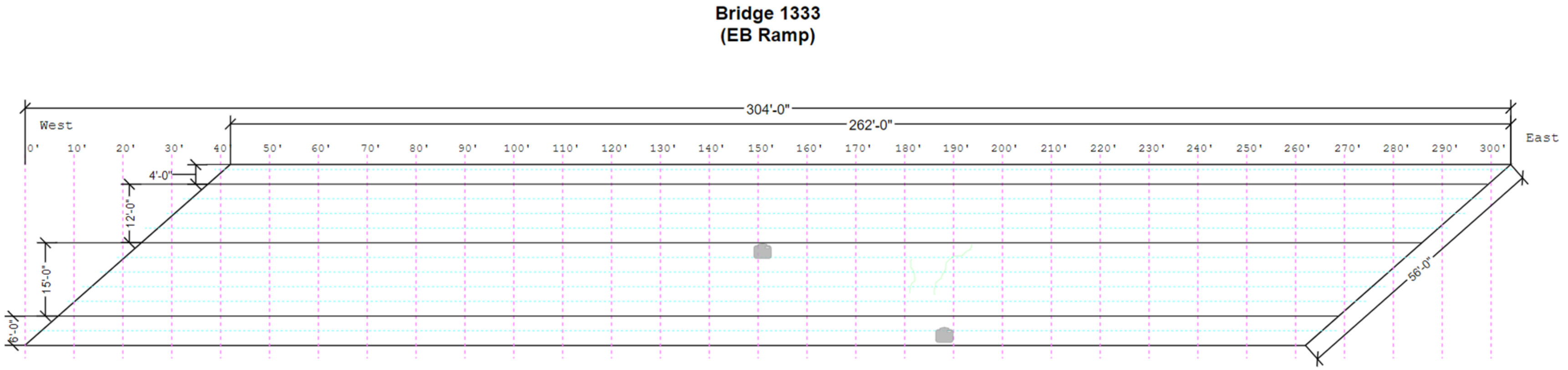

Figure D.12. Field notes for Bridge 1333, 2020 inspection, HFST.

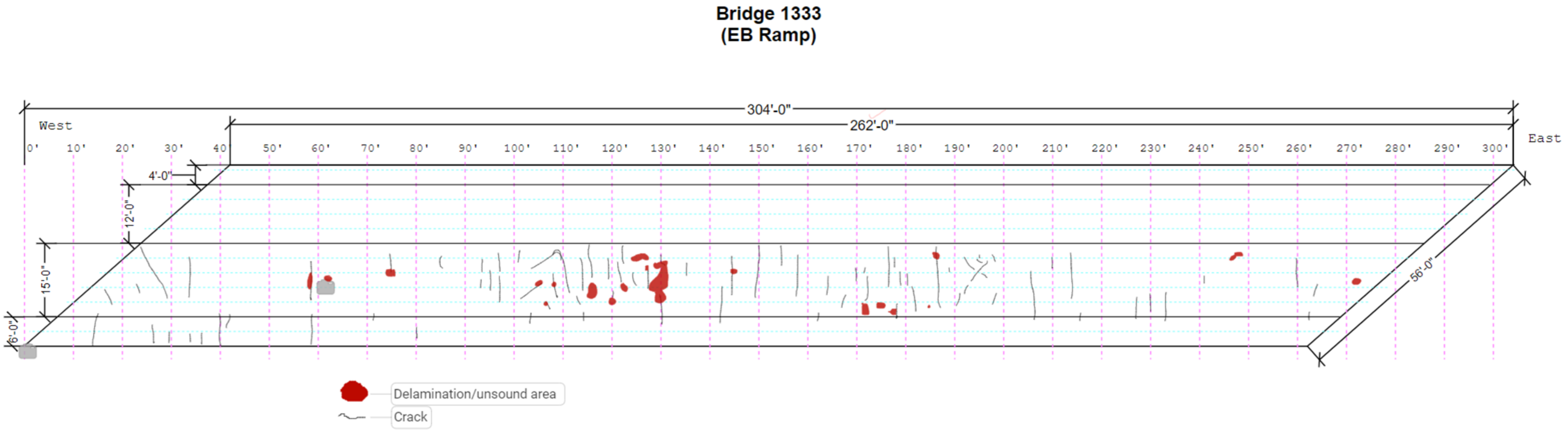

Figure D.13. Field notes for Bridge 1333, 2022 inspection, HFST.

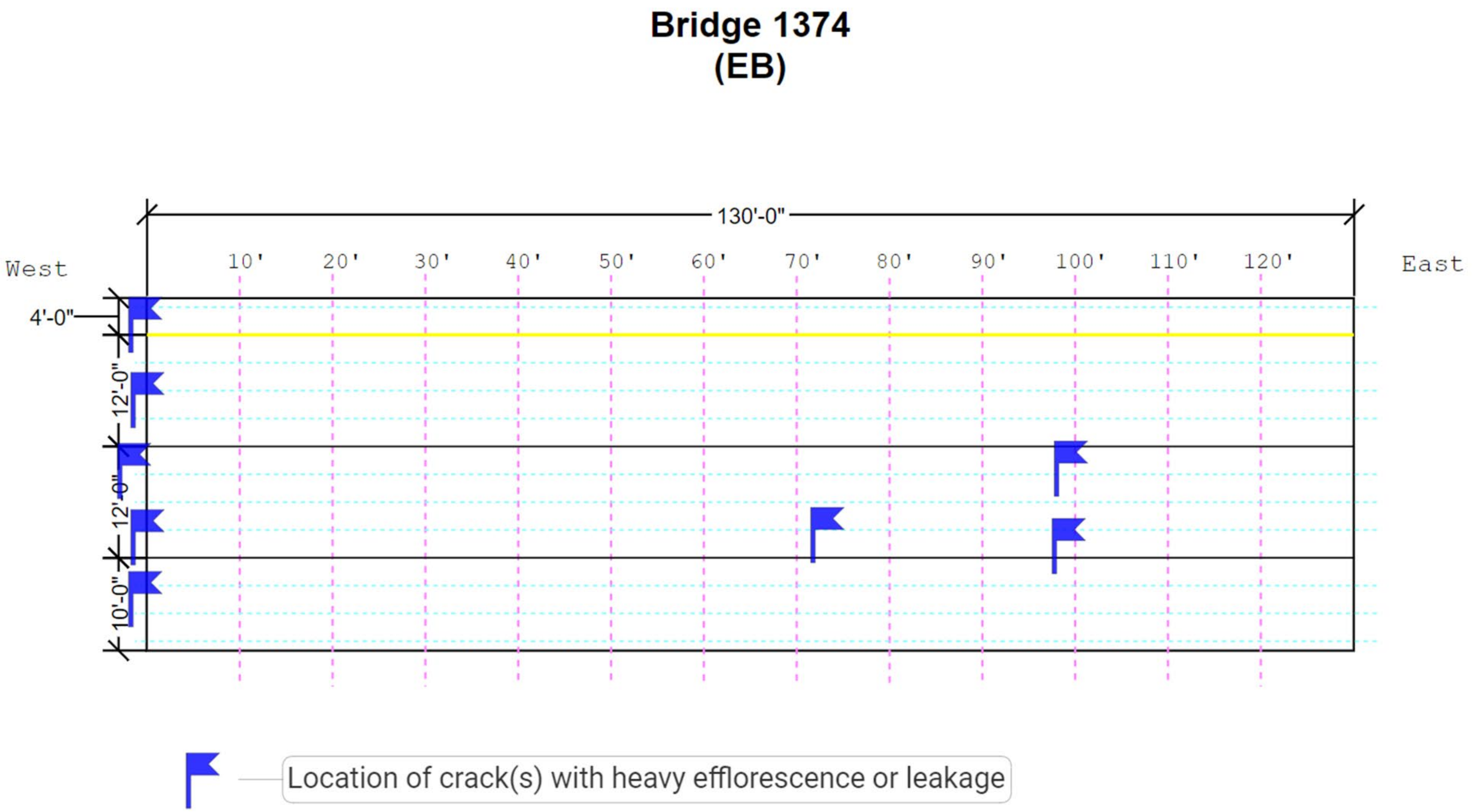

Figure D.14. Field notes for Bridge 1374, 2022 inspection, soffit.

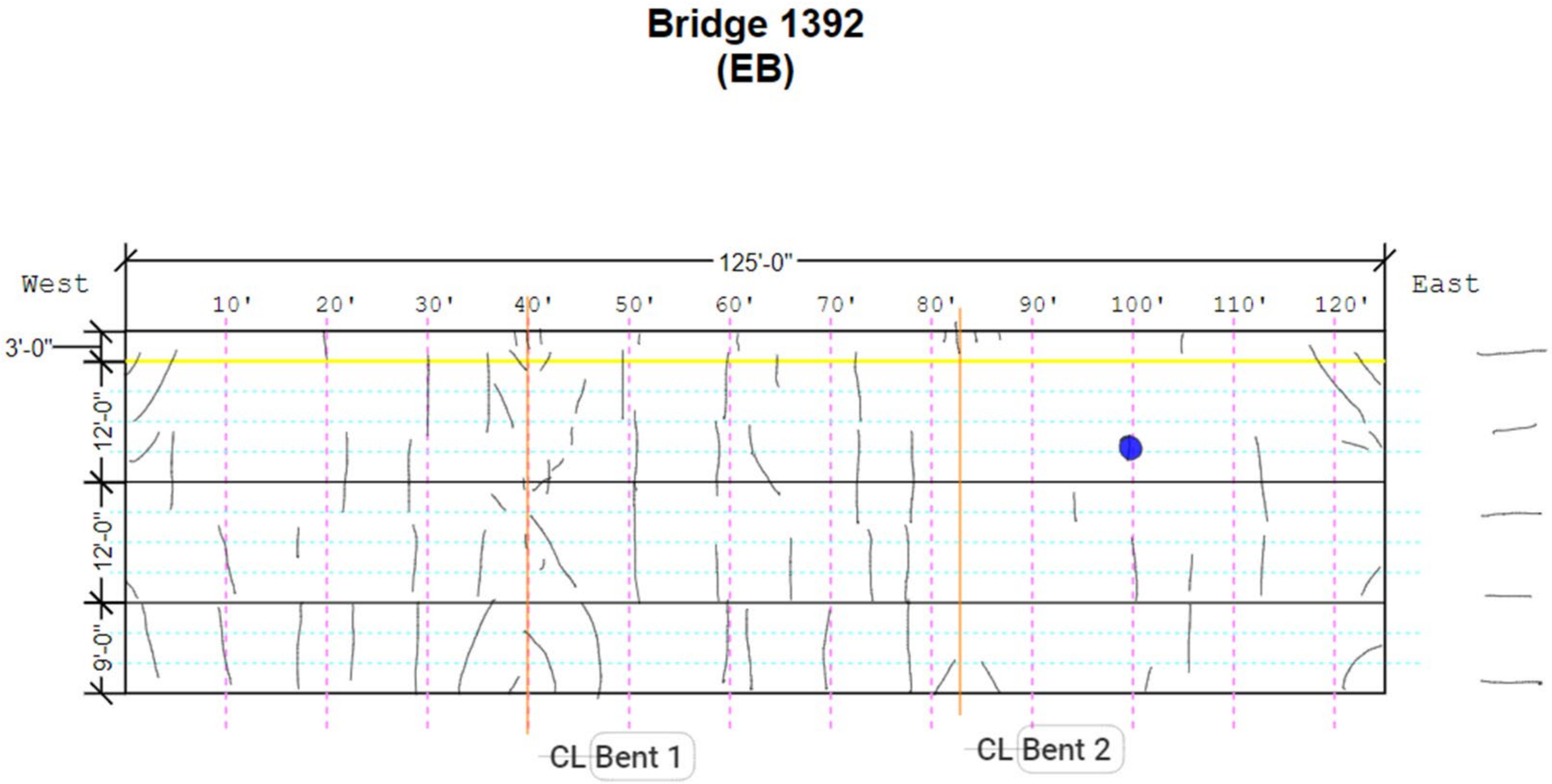

Figure D.15. Field notes for Bridge 1392, 2022 inspection, soffit.

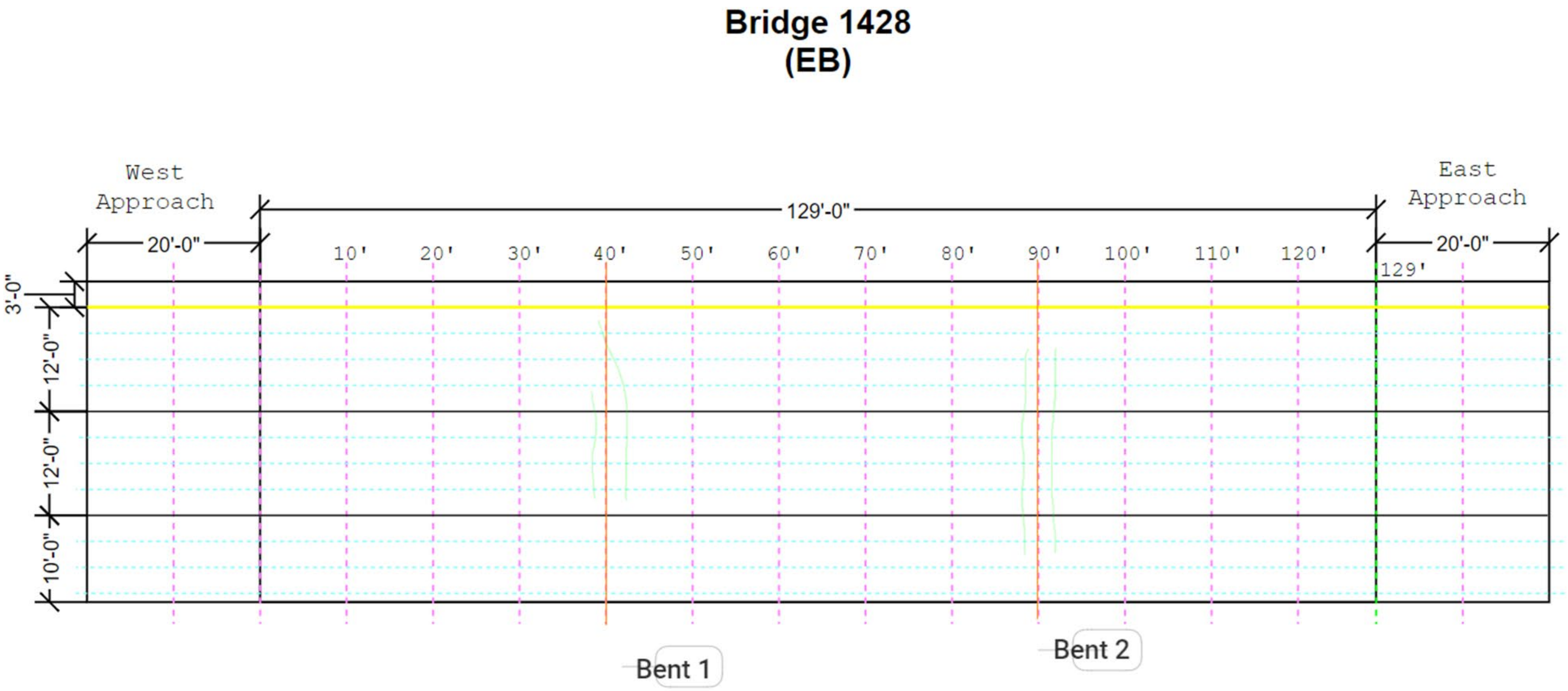

Figure D.16. Field notes for Bridge 1428, 2020 inspection, HFST.

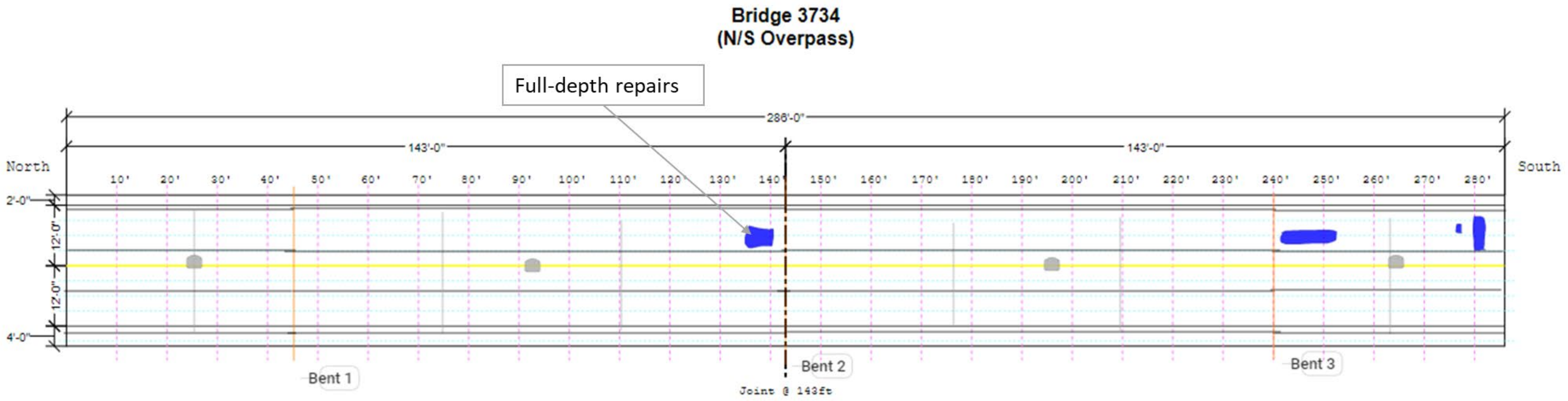

Figure D.17. Field notes for Bridge 3734, 2020 inspection, soffit.

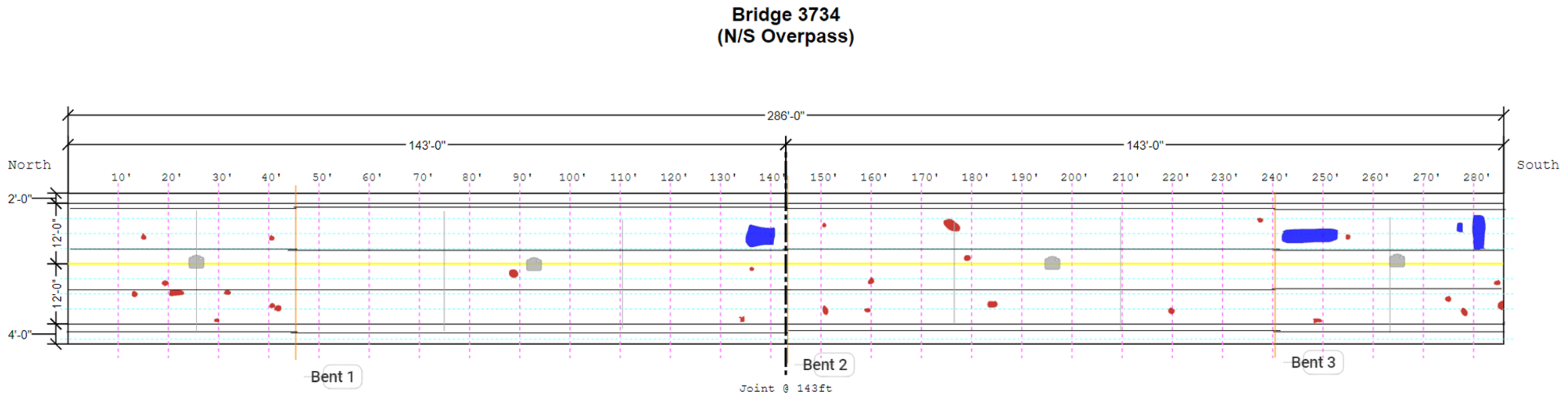

Figure D.18. Field notes for Bridge 3734, 2022 inspection, HFST with full-depth repairs from 2020 inspection shown.

APPENDIX E. LABORATORY TEST RESULTS

Appendices are available on the project web page: https://www.mdt.mt.gov/research/projects/const/evaluation.aspx

APPENDIX F. FULL PETROGRAPHIC REPORT

Appendices are available on the project web page: https://www.mdt.mt.gov/research/projects/const/evaluation.aspx

APPENDIX G. CURRENT POLYMER OVERLAY SPECIAL PROVISION USED BY MDT (DATED 10-08-2020)

Appendices are available on the project web page: https://www.mdt.mt.gov/research/projects/const/evaluation.aspx

APPENDIX H. SKID RESISTANCE TESTING TEST REPORT

Appendices are available on the project web page: https://www.mdt.mt.gov/research/projects/const/evaluation.aspx

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