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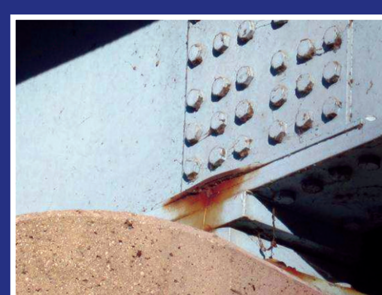
Pack Rust Identification and Mitigation Strategies for Steel Bridges



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16. Abstract
Pack rust or crevice corrosion is a type of localized corrosion. When a metal is in contact with a metal, or even non-metal, the metal starts to corrode, and rust starts to pack in between the surfaces. When significant development of pack rust occurs, it can cause overstressing of bolts and rivets causing them to fail, and it can bend connecting plates and member elements thus reducing their buckling capacity. Thus it is important to mitigate the formation and growth of pack rust in bridges. This study was conducted to determine if pack rust occurs frequently and thereby may pose a problem in the state of Indiana. The study is divided into three primary tasks. The first part of the study involves understanding the parameters involved in the initiation process of crevice corrosion and post-initiation crevice corrosion process. The second part of the study involves reviewing existing mitigation strategies and repair procedures used by state DOTs. The third part of the study involves identifying steel bridges with pack rust in Indiana. Analyses were performed on the data collected from Indiana bridges that have pack rust. This involved finding the components and members of bridges which are most affected by pack rust and finding parameters which influence the formation of pack rust. Pack rust in the steel bridges were identified using the INDOT inspection reports available through BIAS system. The study revealed that good maintenance practices helped in reducing pack rust formation. The study identified locations on steel bridges which have a high probability towards pack rust formation. A mitigating strategy possessing qualities which can show promising results is identified.

17. Key Words
pack rust, crevice corrosion, bridges, mitigation strategies, repair, coatings, inspection

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EXECUTIVE SUMMARY

PACK RUST IDENTIFICATION AND MITIGATION STRATEGIES FOR STEEL BRIDGES

Introduction

Corrosion is a major problem in the infrastructure industry, costing millions of dollars every year for maintenance, repair, or replacement. By improving the coating systems and frequency of application, surface corrosion can be effectively controlled. However, very few studies have been conducted to study the influence of pack rust (crevice corrosion) on steel bridges.

When steel elements of a member are unprotected and in contact with another metal, or even non-metal, the steel usually begins to corrode, and rust (iron oxide) starts to develop, or pack in, between the surfaces. Pack rust is not visible until rust product starts deforming the adjoining members and elements. It can cause overstressing of bolts and rivets, and unchecked rust growth may result in bolt and rivet failure. This will reduce the effective capacity of the connection, or might even cause its failure. The Mianus River bridge collapse is an example of the failure of a connection due in part to pack rust formation.

The average age of the existing steel bridges in Indiana is currently about 50 years, and with the continued aging of the bridge infrastructure, the problem of pack rust is most likely going to increase without proactive intervention. In 2012 INDOT included stripe coating in its painting specifications to mitigate pack rust in new structures. But because this adoption is relatively recent, stripe coating for pack rust mitigation is still in question. This study collected quantitative data on the occurrence of pack rust on steel bridges in the state of Indiana and reviewed mitigation strategies used by other DOTs.

Findings

- Pack rust was found to occur frequently in Indiana. About one-third of the state-owned steel bridges exhibit some form of pack rust. The member element most commonly affected (in terms of numbers) is rocker bearings: 318 of 982 bridges showed evidence of pack rust in the rocker bearings. The second member element to frequently exhibit pack rust is a bolted or riveted splice connection: 214 of 1611 bridges were observed to have developed pack rust in a beam or girder splice connection. The members with the highest percentage of pack rust occurrence are gusset plates and hinge-pin connections: pack rust occurred in both of these components in more than 90% of the bridges with such details. End diaphragms, cross bracings, and beam cover plates were also found to be susceptible; however, the frequency of pack rust occurrence in these members is less than 10%.
- The percentage of observed pack rust occurrence for each district was tabulated. Occurrence in the Greenfield and LaPorte Districts is the least among the six districts in Indiana. The LaPorte District, which experiences the highest amount of annual snowfall and also has the highest salt and brine usage in the state, has a pack rust occurrence of 24%, which is less than half of that observed in the Fort Wayne District. There are multiple possible reasons for this observation. One factor that may play a large role is that the LaPorte maintenance crews annually wash the decks and bearings of every bridge using water jets to remove dirt, debris, and salts.
- The occurrence of pack rust in girder and beam splices of bridges that intersect a water body is higher than that of bridges that intersect roads and railroads. The percentage of bridges with pack rust in the splice of exterior beams is higher than that of bridges with pack rust in the splices of interior beams. The study found that it takes 12 years on average after painting a bridge (i.e., re-coating) for crevice corrosion to start in the gap between the members and the splice plates to exhibit visible rust bleeding from the splices. The use of spot painting or recoating at a frequency of less than 12 years may help to minimize pack rust formation. From the point of initiation, it would then take an additional 20 years to reach a very severe pack rust condition.
- With the help of images present in inspection reports, it was observed that the edge distance and the initial pretension in the bolts play a major role in preventing pack rust in splice connections and other connections.
- Stripe coating as a pack rust mitigating strategy is the most popular technique utilized, with 24 state DOTs recommending it in their painting specifications. Thirteen states recommended caulking and 8 states recommended the use of penetrating sealers. Oregon is the only state DOT that outlined a method to repair members affected by pack rust.
- Experimental studies showed that stripe coated connections with the bottom crevice un-caulked experienced the least amount of corrosion and minimum pit depth for new structures. A second series of specimens involved plates that were corroded, cleaned, assembled, and then stripe coated and caulked: caulk placed on all sides was found to produce the best results.

Implementation

- The use of small edge distances with properly tightened high-strength bolts will keep material in firm contact and minimize crevice openings. The use of bolt stagger in new splice connections should be avoided.
- Current INDOT provisions for stripe coating of new structures should be retained. Further study should be done to investigate the effectiveness of stripe coating and the need to modify the number of stripe coats utilized.
- Pack rust formation can be minimized in splice plate details where no pack rust has been detected if the connection region is cleaned and a stripe coat is applied along the crevice at a frequency of no more than 12 years. The opening between the flanges can be sealed with a suitable filler material to prevent moisture entry. If rust bleeding is observed in splice connections, use of an alkaline penetrating sealer appears to be the best option.
- If caulk is used to seal crevices, rust, debris, and salts should be removed and the surfaces cleaned before caulking the crevice. Otherwise they should not be caulked. Caulking an active crevice corrosion cell will likely accelerate the corrosion process.
- Penetrating sealers that are alkaline and have the appropriate viscosity to penetrate into crevices show promising results in mitigating pack rust. The crevice should be cleaned by mechanical tools or high-pressure water jets before applying penetrating sealers. Further study of these sealers should be considered to establish whether they should be used regularly in Indiana.
- Pressurized water jet washing appears to be an effective maintenance practice that reduces the chances of pack rust occurrence in bearings.

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1. INTRODUCTION

Corrosion is a major problem in the nation's infrastructure. It is a problem for both structural steel and reinforced concrete structures. Interstate highway construction following World War II generated a sudden demand for bridges to have unobstructed traffic flow. Structural steel was one of the primary materials of choice for bridges in those years.

Figure 1.1 shows the statistics for the number of bridges built in Indiana over several decades. The bridges were designed and built for a service life of about 50–70 years. It is clear that most of the bridges from the 1950s to 1960s are nearing the end of their original intended service life, with many of these bridges needing maintenance and repair. Corrosion is a time dependent process and half a century is a sufficient amount of time for steel to corrode very severely. General corrosion and pitting corrosion are a concern causing section loss and a decrease in load capacity. About 15% of the structurally deficient bridges are deficient because of corrosion (Koch, Brongers, Thompson, Virmani, & Payer, 2002). General and pitting corrosion are visible on the surface. Crevice corrosion, however, is often not visible until it becomes very severe, and can lead to serious problems.

Pack rust has been a topic of research interest in the chemical industry. Much research has been conducted to study the behavior of crevice corrosion in stainless steel. The Naval Research Laboratory has conducted some research on iron for crevice corrosion. However, there is limited research being conducted by the bridge industry, although, there is a documented case of a bridge collapse due to pack rust; not directly, but indirectly. The Mianus River bridge collapse occurred on June 28, 1983 in Greenwich, Connecticut. It was discovered that pack rust displaced the hanger bar of a pin and hanger assembly by 1½ inches out of plane from the girder web. This lead to an off-center load on the pin and the ultimate failure was due to the fatigue fracture of the pin see Federal Highway Administration (FHWA, 2015) and National Transportation Safety Board (NTSB, 1984).

1.1 Objective

The study involves three major tasks:

1. Literature review:
   a. Understanding pack rust, or crevice corrosion.
   b. Understanding how crevice corrosion initiates and propagates.
   c. Collecting relevant existing research on pack rust.
2. Reviewing existing mitigation strategies and repair procedures utilized by other state DOTs.
3. Documenting the occurrence of pack rust in Indiana bridges. This task involves answering the following questions:
   a. Does pack rust occur frequently in the state of Indiana and, if so, is it a problem?
   b. Which components or members of the bridge are prone to crevice corrosion?
   c. What parameters influence the formation of pack rust?

1.2 Scope

To identify the bridges in Indiana with pack rust, bridge inspection reports available through the state's Bridge Inspection Application System (BIAS) system were used. It should be emphasized that all pack rust identification was done from a review of the inspection reports and not by site inspections. A database of bridges with pack rust was created and stored in Microsoft Excel. Statistical analyses were performed on the data to identify the parameters and factors which influence the formation of pack rust.

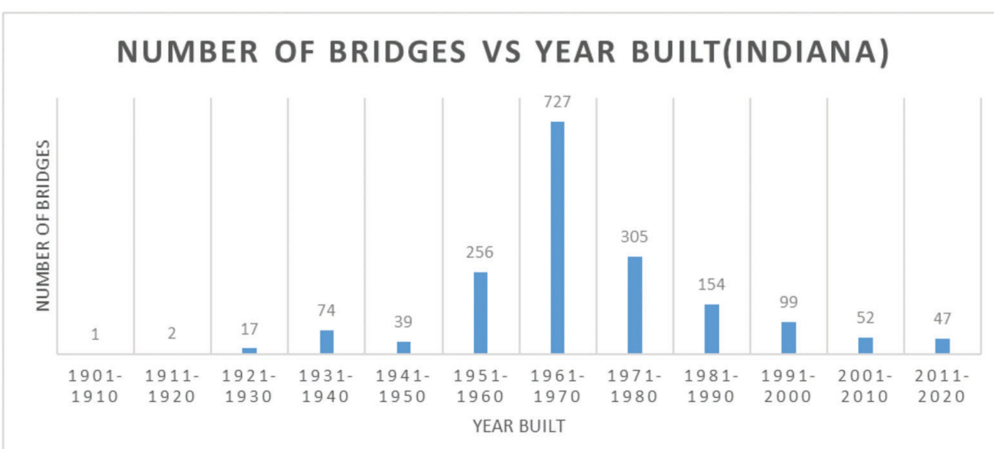


Figure 1.1 Distribution of the steel bridges built in Indiana.

2. LITERATURE

2.1 Corrosion

Corrosion is the process of material degradation (both metallic and non-metallic) due to the chemical reaction with the environment. Nature does not store metal in its pure form; it exists in the form of compounds (most common form oxides). Corrosion reactions proceed forward without application of any external energy and metal reaches its stable state. Extraction of metal from its ore needs external energy. Hence, corrosion processes are metal extraction processes in reverse (Fontana, 1987).

2.2 Cost of Corrosion

A study conducted from 1999 to 2001 shows that a total of $276 billion is spent on corrosion-related issues in commercial, residential and transportation sectors. This cost is approximately 3% of the U.S GDP in the year 1998 (U.S GDP in 1998 was $9.09 trillion). The cost of corrosion in the infrastructure industry amounts to $22.6 billion, out of which $8.3 billion is spent on highway bridges. It is observed from the Figure 2.1 the cost of corrosion in highway bridges is about a third of the total cost in the infrastructure industry (Koch et al., 2002).

The total number of bridges in the U.S. is approximately 583,000, of which 200,000 are steel, 235,000 are conventional reinforced concrete, 108,000 are pre-stressed concrete, and the remaining are made of other construction materials. Corrosion is the reason for approximately 15% of structurally deficient bridges (Koch et al., 2002). The estimated annual direct cost is $8.3 billion. Figure 2.2 shows a further breakdown of the cost of corrosion in highway bridges. The majority of the cost, about $3.8 billion, is due to the replacement of the structurally deficient bridges. Maintenance and capital costs for concrete bridge decks and concrete substructures are the other two segments where a total of $4 billion is the cost of corrosion. The least amount spent of the categories identified is for maintenance painting of steel bridges. Painting is an important task because it helps in extending the life of the structure by preventing it from developing corrosion. The total cost estimated does not include additional user costs due to traffic delays, long detours that add to more fuel consumption, and wear and tear of vehicles. (Koch et al., 2002).

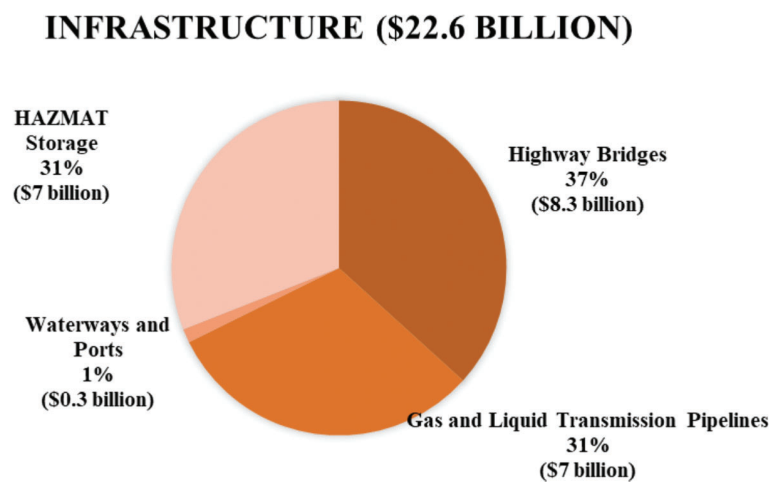


Figure 2.1 Cost of corrosion in the infrastructure industry (Koch et al., 2002).

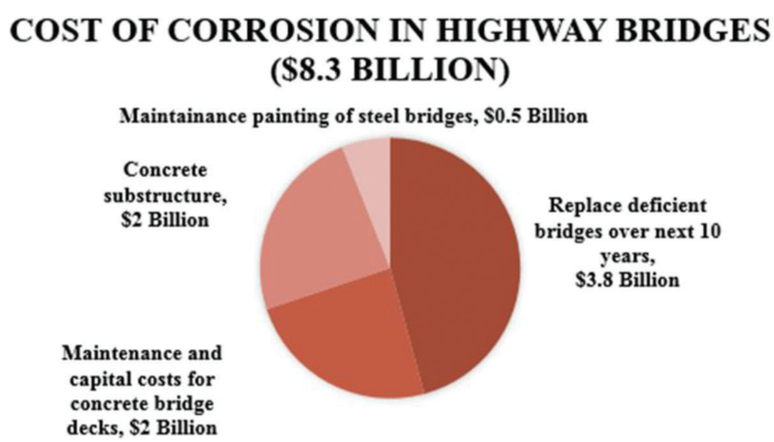


Figure 2.2 Cost of corrosion in highway bridges (Koch et al., 2002).

2.3 Types of Corrosion

Water is often blamed for corrosion. However, corrosion can still be observed in dry conditions where moisture is absent. High-temperature furnace gases can also cause corrosion in steel (Fontana, 1987). This type of corrosion is classified as dry corrosion, and one that occurs due to the presence of water is classified as wet corrosion. Wet corrosion needs an aqueous solution which serves as a path for the ions to flow and complete the charge flow circuit.

Different forms of corrosion identified by Jones (1996) are as follows:

1. Uniform corrosion
2. Galvanic corrosion
3. Crevice corrosion
4. Pitting corrosion
5. Environmentally induced cracking
6. Hydrogen damage
7. Intergranular corrosion
8. Dealloying
9. Erosion corrosion

Bridges experience all the forms of corrosion from 1 to 5, but hydrogen damage, intergranular corrosion, dealloying corrosion, and erosion-corrosion are not observed in bridges due to the nature of the material used in bridges and the conditions required to cause these forms of corrosion. In this current study, the focus will be on crevice corrosion.

2.4 Crevice Corrosion

Crevice corrosion is the localized form of corrosion which takes place inside the crevice formed by the contact between two metal surfaces or the surface between a metal and non-metal. A portion of the metal which is in contact develops corrosion. Locations on bridge where crevice corrosion is commonly observed include connection details such as splice plates, gusset plates, and the surfaces between bolt/rivet head and steel plate, and bolt shank and plates. These are the examples of contact between steel and steel. Deposits of sand and dirt on the metal surface can also cause crevice corrosion. The deposits may also act as a shield and the corrosion chemistry depends on the porosity of the deposit.

2.4.1 Mechanism

The fundamental mechanism behind crevice corrosion is still being studied. There are two major theories for the mechanisms of crevice corrosion. The first theory is the traditional theory based on the occluded chemistry change, or the critical crevice solution (CCS), and the second theory is based on the ohmic drop or the IR model. Both the theories are not able to prove all the observations seen in the actual crevice corrosion process.

2.4.1.1 Theory 1. Initially, there is ambient oxygen and aqueous solution present both within and outside the crevice. As the process of uniform corrosion starts, minute local anodic and cathodic sites are formed. The redox reactions taking place inside and outside the crevice are:

$$\text{Anode: } M \rightarrow M^{+n} + ne^-$$

$$\text{Cathode: } 2H_2O + 4e^- + O_2 \rightarrow 4OH^-$$

At the start, the kinetics of the reactions are the same throughout as shown in Figure 2.3. The cathodic reaction consumes oxygen. As the corrosion continues, oxygen concentration inside the crevice starts to drop and complete replenishment of the oxygen from the outside is restricted due to the geometry of the crevice. Due to the lack of oxygen in the crevice, the cathodic reaction gets suppressed and the only reaction taking place inside the crevice is the anodic reaction. As the anodic reaction in the crevice increases, the rate of cathodic reaction on the unshielded surface also increases to

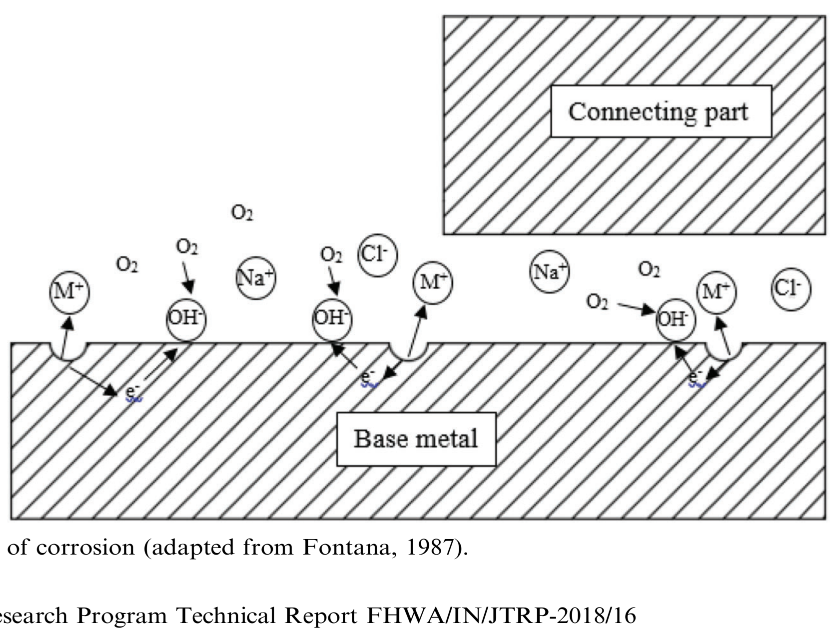


Figure 2.3 Initial stage of corrosion (adapted from Fontana, 1987).

balance the charge flow and acts as a cathode. Now that the anodic reaction is localized inside the crevice and the cathodic reaction on the non-shielded surface, the condition is set for the localized corrosion.

With the formation of localized anode and cathode at different locations, a potential gradient is developed between the shielded and non-shielded surfaces. If chlorides are present in the bulk solution, the potential gradient causes the chloride ions to travel from the bulk solution to the crevice solution. Metal ions hydrolyze in the presence of the water and produce hydrogen ions. Increase in the hydrogen ions leads to an acidic crevice solution. Accumulation of the chloride ions and decrease in the pH inside the crevice creates a severe corrosive environment. Due to the formation of the hydroxyl ions in the bulk solution the pH increases, and solution becomes alkaline. The pH of the crevice solution gets stabilized at around pH 3 – pH 4 since hydrolysis is thermodynamically unfavorable below this pH (Pickering & Frankenthal, 1972).

Figure 2.4 shows that as the crevice corrosion first develops the chloride ions start to travel inside the crevice. The concentration of hydrogen ions increases inside the crevice and concentration of hydroxyl ions increases in the bulk solution. The metal ions produced by the anodic reaction starts to move outside the crevice and on their way outside becomes deposited at the mouth of the crevice as rust product, as shown in Figure 2.5. Figure 2.5 also shows how acidic and chloride rich crevice solution which gets trapped by the rust deposited at the mouth of the crevice creates a severe corrosive environment.

Research at the Naval Research Laboratory demonstrated that when the pH of the crevice solution is set to pH 2, it increases to pH 4 (Brown, Fujii, & Dahlberg, 1969). The crevice corrosion process is unstable at pH levels lower than 4.

The traditional mechanism fails to explain the crevice corrosion in cases where no chlorides are present or in the buffered solution where the pH remains constant. (DeForce, 2010)

2.4.1.2 Theory 2. The second theory is based on the IR voltage developed by Pickering and Frankenthal (1972). They observed that the calculated IR drop or potential drop within the crevice is usually much smaller than the measured voltage drops, where I is the current flow from the cathode towards anode and R is the resistance provided by the aqueous solution. A potential difference exists between the anode and the cathode in an electrochemical cell. In case of crevice corrosion where the anodic site is separated from the cathodic site, the potential difference or the voltage drop is in the range of 10^2 to 10^3 mV (Pickering, 1993).

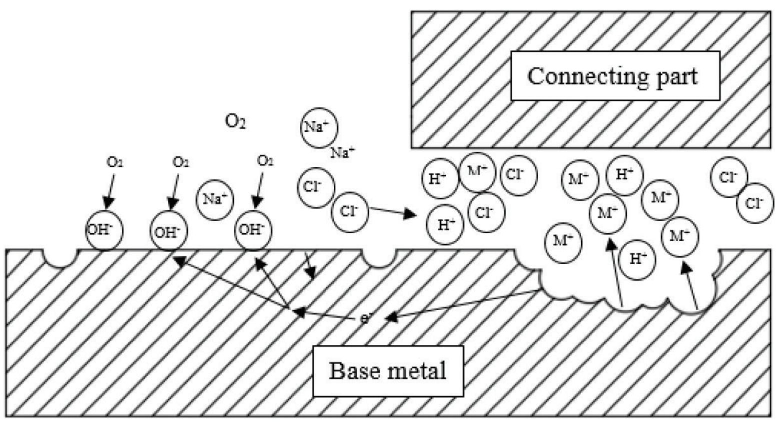


Figure 2.4 Corrosion at a later stage (adapted from Fontana, 1987).

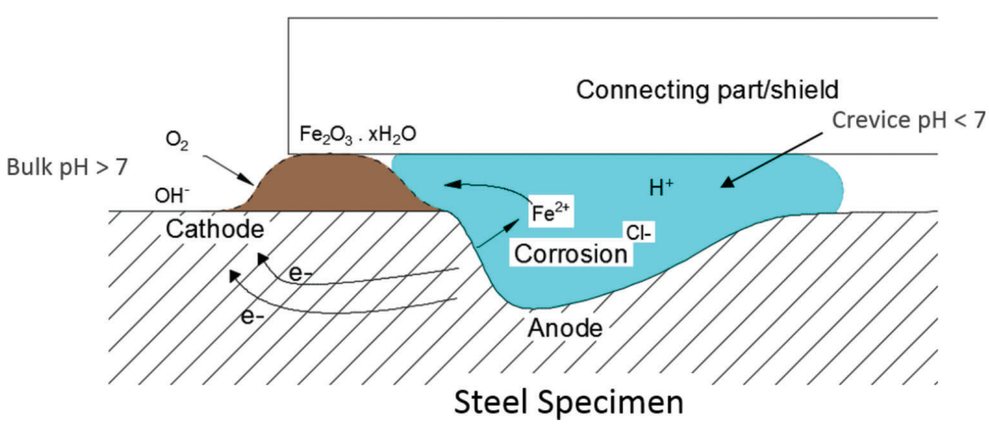


Figure 2.5 Rust deposit at the mouth of the crevice and acidification of crevice solution.

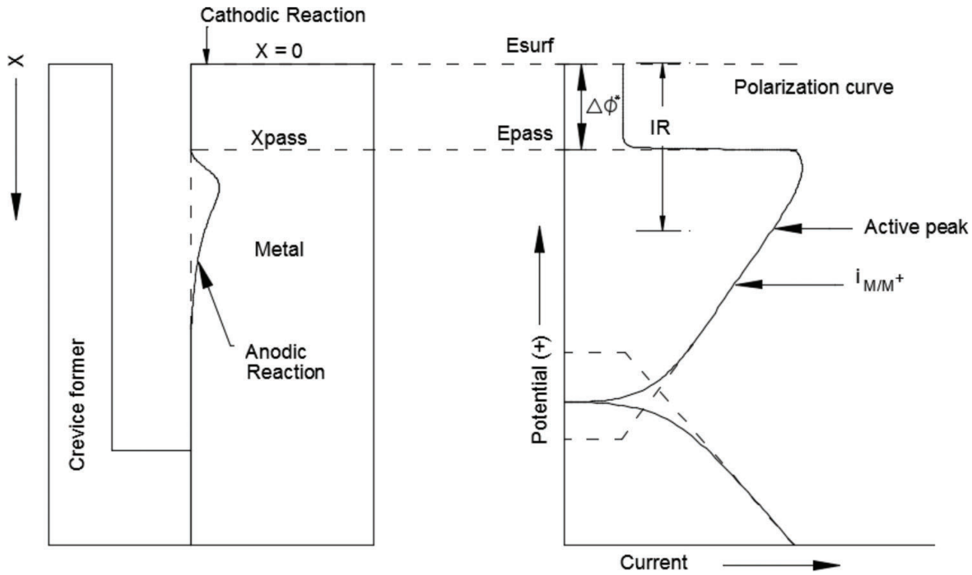


Figure 2.6 Schematic of metal with crevice (*left*) and its matching polarization curve (*right*) (Nystrom et al., 1994).

The crevice corrosion process is explained in terms of an electrical circuit, as shown in Figure 2.6. The external surface has a potential E_{surf} which has a noble potential and thus low dissolution rate. The current, I, flows from the outside surface into the crevice through the aqueous solution of resistance, R, and produces a voltage drop in the crevice. The potential, E_x, at a distance x from the mouth of the crevice can be calculated as by Equation **2.1** (Pickering, 1993):

$$E_X = E_{surf} - IR \tag{2.1}$$

Within the crevice, E_x shifts to a less noble potential. The metal surface from the mouth of the crevice to a distance x_{pass} remains passive. At x_{pass} the potential of the metal is E_{pass}, and at this point anodic current peaks to a maximum value. Beyond x_{pass} active dissolution occurs and is known as an active loop. For crevice corrosion to occur the potential drop should be greater than the difference in the potential at the surface and $E_{pass,}$ and is denoted as $\Delta\Phi^*$. Therefore, $IR > \Delta\Phi^*$ is the condition for the crevice corrosion to occur (DeForce, 2010). Figure 2.6 on the left shows an experimental setup used at Pennsylvania State University to evaluate crevice corrosion, and on right the crevice polarization curve is depicted. The shape of the corroded region closely resembles the polarization curve (Nystrom, Lee, Sagues, & Pickering, 1994).

Although the IR mechanism can explain the crevice corrosion in a buffered solution (i.e., a constant pH) and in the absence of chlorides, which the traditional mechanism was not able to do, the IR mechanism does not provide a complete understanding of the behavior of crevice corrosion. Further studies are required to find the relation of corrosion kinetics with the size and shape of the crevice. The IR mechanism comes into picture only after crevice corrosion has started. It is not able to explain the initiation process of the crevice corrosion. This mechanism explains the crevice corrosion behavior in metals which shows passivation during anodic polarization such as stainless steel, titanium and depending on the electrolyte, also iron and carbon steel.

Various tests with changing parameters were conducted at Pennsylvania State University to observe the behavior of crevice corrosion. The parameters included pH, surface potential, the presence of inhibitors and chlorides in solution, the effect of temperature, crevice width, and depth.

The experiments at Pennsylvania State University were mainly conducted on stainless steel, which is not the material used in the bridge industry. The experiments were also focused on the environments in the chemical industry. Some experiments conducted on iron were performed on bare specimens and the behavior would be completely different if those were conducted on coated specimens.

2.4.1.3 Contradiction in both the mechanism. According to IR mechanism the corrosion is maximum near the mouth of the crevice whereas according to the traditional mechanism, maximum corrosion takes place in the deepest point in the crevice. Clearly, further basic research is needed to understand the mechanics of crevice corrosion, and how it varies for different materials.

2.5 Other Research

The research conducted by Naval Research Laboratory tested some of the parameters involved in crevice corrosion in iron specimens. One of the parameters tested was the crevice height. The relationship of cathodic current and overvoltage by changing crevice height was investigated. The experiments were conducted for 5 mils, 10 mils, 20 mils and 125 mils crevice height. The tests showed that with the decrease in the crevice height the cathodic current decreases and hence the corrosion rate is reduced. It was seen that the current for the crevice heights in the range 5 to 20 mils remains constant. This is thought to be because the thickness of the oxygen diffusion layer in a 0.5N NaCl solution is calculated to be 20 mils (McCafferty, 1974).

The experiments conducted by the Naval Research Board were based on the cathodic polarization, but the experiments conducted at Pennsylvania State University were based on the anodic polarization and thus there will be variation in the results. The variation is because metals do not show passivation in cathodic polarization.

To prevent crevice corrosion in bridges carbon steel needs to show passivation behavior with large $\Delta\Phi^*$ during anodic polarization. A plasma nitride treatment on carbon steel forms a nitrogen solid solution layer on the surface which increases the corrosion resistance of the carbon steel. The anodic polarization curves show a passivation behavior (Chiba, Nagataki, & Nishimura, 2016). A plasma nitride treatment on the components of the bridge which will be in contact can prevent crevice corrosion. However, this method is expensive and would be very uneconomical at the initial phase for application on bridges. It cannot be recommended without further study on its life cycle cost of maintaining steel bridges over their entire design life.

3. MITIGATION AND REPAIR STRATEGIES

3.1 Need for Mitigation and Repair Strategies

It is clear from Figure 1.1 that much of the steel highway bridge infrastructure in Indiana is getting old and has served for a significant percentage of its service life. Figure 2.2 also shows how much money is spent nationally on highway bridges because of corrosion. The highway bridge industry has expended considerable time and resources towards extending the life of bridges by improving the coating (painting) system. The effort was mainly to prevent surface corrosion. However, crevice corrosion was not specifically considered until cases of bolts failure, rivet failure, and excessive plate distortions were observed.

Observing the current condition and planning for the future calls for research and development of mitigation strategies for newly built bridges and a pro-active repair procedure for existing bridges. The repair strategies should be able to extend the useful life of existing steel bridge members or elements before they experience a significant strength reduction or total failure.

3.2 Strategies Used by DOTs

Painting specifications of all 50 DOTs were reviewed in search of any provisions specified by the DOT to mitigate pack rust in new bridges or repair procedures for pack rust in existing bridges. Four primary mitigating and repair strategies were found in the coating requirements to tackle pack rust in new and existing bridges. These are as follows:

1. Stripe coating
2. Caulking
3. Penetrating sealers
4. Backer rods

3.2.1 Stripe Coating

A stripe coat is a coat of paint which is applied at the edges, corners, crevices, seams, interior angles, junction of joining members, weld lines or other surface irregularities. The underlying paint coat thickness (primer coat or intermediate coat) at all these locations is less than the paint thickness on the flat surface due to the nature of geometry and the surface tension of the paint film as shown in Figure 3.1. The green film in Figure 3.1 is the paint which is applied to the entire surface, and the blue film is the stripe coat applied on the undercoat system. This additional coat of paint at these locations increases the paint thickness and thus decreases moisture permeability and increases corrosion resistance. The stripe coat at the crevices will significantly increase the paint film thickness and reduce the moisture penetration into the crevice as seen in Figure 3.1. Preventing moisture from entering into the crevice is

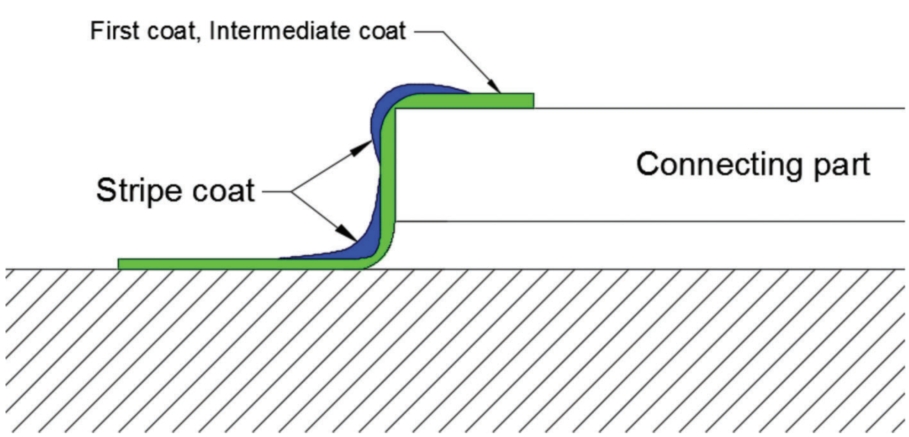


Figure 3.1 Paint film and stripe coat thickness.

the motive behind the stripe coat application to prevent crevice corrosion.

The use of a stripe coat was the most common method employed by the DOTs in their effort to mitigate pack rust. There are 24 out of 50 states that require some form of stripe coating in their painting specifications. The paint used for the stripe coat is same as that of the undercoat or the one which will be provided after the application of the stripe coat in some cases. There are multiple application sequences of stripe coat with the painting system used by various DOTs. Some DOTs recommend one stripe coat that is of either primer coat or intermediate coat, while other DOTs recommend up to three stripe coats. In general, it is recommended that the tint of the stripe coat should be in contrast with that of the undercoat, since this makes it easier for the field inspector to inspect the striped location.

The probability of getting cracks in the paint film at the crevices are high if the paint film is too thick. The cracks will also occur due to the differential thermal expansion of adjoining components. Environmental factors and time will also impact the paint and cause cracking. The development of cracks will allow the entry of moisture inside the crevice and promote crevice corrosion. Although there is no particular period established as to when the paint starts to form cracks along the crevices, it is evident that cracks will eventually form. Stripe coat should stay in place and mitigate crevice corrosion until the bridge is repainted after 30 years, which is roughly a typical painting cycle. All this gives an impression that stripe coating cannot serve as a one-time, long-term mitigation strategy but, stripe coating will definitely delay the pack rust formation in steel bridge members.

3.2.2 Caulking

Caulk is a waterproof filler used to seal the crevices. Caulking is done to prevent the entry of the water into the crevice. The concept behind using caulking to prevent pack rust is same as that behind stripe coating, i.e. preventing the entry of the water into the crevice.

There are 13 states which recommend caulking in their painting specification as a mitigation strategy for pack rust. Caulking tends to crack over time due to various reasons, such as variations in the atmospheric conditions and differential thermal expansion. Cracks will allow moisture into the crevice. It can be considered a good option for new structures because it might delay or extend the process of crevice corrosion by a few years. However, caulking may be a bad choice where pack rust already exists and caulking is applied without any treatment. Ballinger and Senick (2003) found that if an ongoing crevice corrosion cell is sealed the corrosion process accelerates by 400 fold. On the contrary, Evans (1948) believes that if the rivets or bolts holding the plates are sufficiently strong, the crevice will seal itself, and the action will stop. However, it is possible that the corrosion process may start again and fail the rivets or bolts. Having the knowing that the crevice corrosion process creates an acidic environment inside the crevice, it is likely that sealing the crevice without cleaning, and perhaps treatment, will serve no good in mitigating the pack rust formation, if not accelerating.

3.2.3 Penetrating Sealer

The name penetrating sealers indicates that they have a viscosity sufficient to penetrate the crevice. The crevice corrosion reaction creates an acidic environment inside the crevice, so it is recommended that the penetrating sealers be alkaline to neutralize the acidic crevice solution. The penetrating property and the alkaline nature makes a penetrating sealer suitable for use in both new structures and in existing bridges experiencing pack rust. Penetrating sealers are a good option for new structures because the surfaces which will be in contact after the erection of bridge have often only have a primer coat on them. The penetrating sealer gives the metal surface an additional protective layer. Alkaline penetrating sealers are extremely helpful for use in crevices with active crevice corrosion because of its ability to neutralize acidic crevice solution.

According to the IR mechanism the crevice corrosion takes place at the depths where the active loop of anodic polarization curve exists as shown in Figure 2.6. Assume that x_{active} is the distance from the mouth of the crevice to the point where active dissolution ends, and L is the depth to which penetrating sealer can penetrate as shown in Figure 3.2. For penetrating sealers to work effectively in preventing crevice corrosion, L should be greater than x_{active} ($L > x_{active}$).

There are eight states which recommend penetrating sealers in their painting specifications. Three states mention specific requirements for penetrating sealer, and those are:

- 100% solid rust penetrating sealer, Delaware
- Calcium sulfonate rust penetrating sealer, Missouri
- Epoxy penetrating sealer, New York

3.2.4 Backer Rod

Backer rods are flexible polyethylene or polypropylene foam rods. These are usually used in expansion joints. Backer rods are typically stipulated when the crevice gaps are large. Washington and Oregon DOT recommend the use of backer rods when the crevice height is greater than ¼ inch. After inserting the backer rod into the gap, a sealant or caulk is applied on top of it. Before using a backer rod for an existing structure, the crevices should be cleaned using a high-pressure waterjet or cleaned mechanically, and any active corrosion should be neutralized.

Figure 3.3 shows a schematic diagram for the backer rod and sealant application. When the plates have bent due to pack rust, the gap between the plates often gets large. As the gap is large, backer rod proves advantageous in sealing it and preventing future moisture entry.

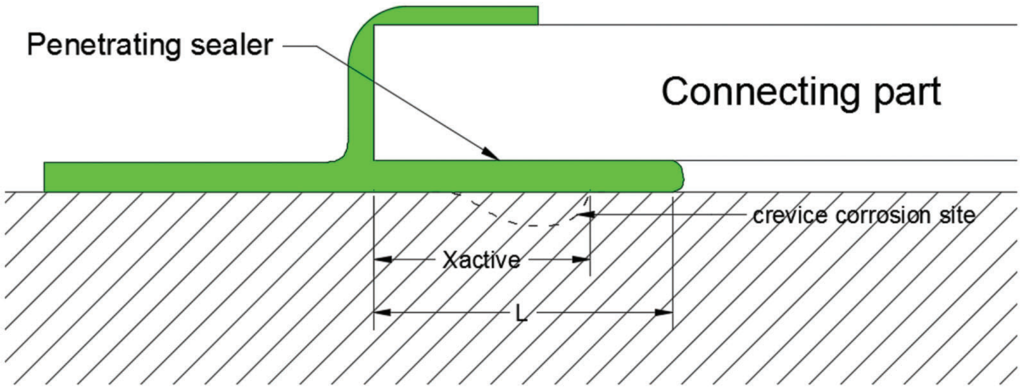


Figure 3.2 Schematic diagram for penetrating sealers.

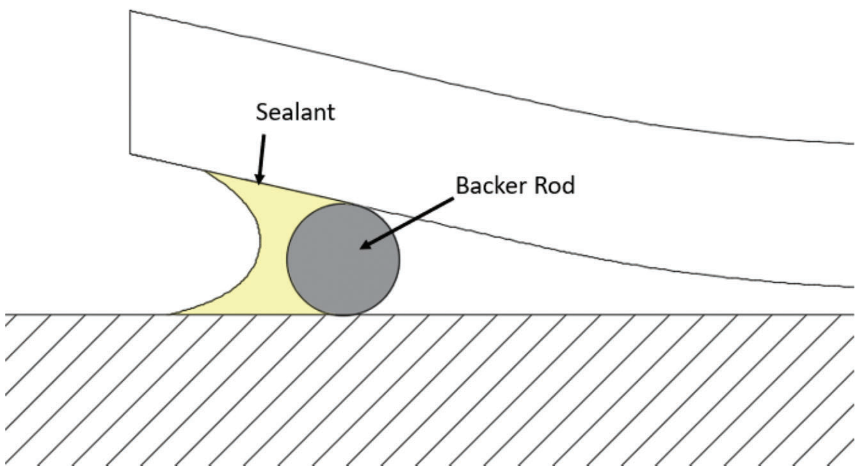


Figure 3.3 Schematic diagram for backer rod and sealant.

TABLE 3.1
Summary of the states using mitigation strategies for pack rust

Stripe coat (24 states)	Caulking (13 states)	Penetrating sealers (8 states)	Backer rod (2 states)
Alabama, California, Delaware, Florida, Georgia, Illinois, Indiana, Iowa, Louisiana, Maryland, Massachusetts, Minnesota, New Jersey, New York, North Carolina, Ohio, Oregon, Pennsylvania, South Dakota, Texas, Virginia, Washington, West Virginia, Wisconsin	California, Delaware, Florida, Indiana, Iowa, Louisiana, Maryland, Missouri, Ohio, Oregon, Tennessee, Washington, West Virginia	Delaware, Illinois, Iowa, Louisiana, Missouri, New York, Texas, Washington	Oregon, Washington

Table 3.1 lists the mitigation strategies used by specific states. Stripe coating is the most widely adopted mitigation strategy followed by caulking, penetrating sealer and backer rod. Table A.1 in the Appendix gives further details on the limits on the size of the crevice when caulking is required, the sequence of stripe coat application, type of penetrating sealer specified by state and comments.

3.3 Repair Procedure by Oregon DOT

The Oregon Standard Specifications for Construction (ODOT, 2015) outlines a method that they use to repair the members affected by pack rust. The process involves heating water-saturated pack rust to a minimum of 250°F and a maximum of 400°F and removing the pack rust by mechanical cleaning. There is no elaborate explanation on the mechanical cleaning in the specifications. Hammering the connection plate affected by pack rust with a rivet gun using a buffer plate can be considered as a means of mechanical cleaning. The efforts of Lansing Community College on restoration and preservation of historic metals for pack rust shows a similar process of repair, the only variation in the process is that the pack rust was not water saturated before hammering the plates with the rivet gun (Lansing Community College, 2009).

Oregon DOT recommends stripe coating the surface with primer coat and intermediate coat. Filling and sealing the crevices between structural shape and plates is recommended if gap cannot be filled with coating material. Backing material is recommended followed by caulking if gap is greater than ¼ inch.

3.4 Industry Effort

Experiments were conducted by Shoyer, Ault, McDonagh, and Prazenka (2018) to test the effectiveness of stripe coating and caulking on new steel and weathered or partially corroded steel with various combinations. The weathered steel were cleaned following SSPC SP-10 and SSPC SP-11 specifications. The new steel specimens were tested for priming before assembly of the plates and after assembly, which created the crevice. The combinations used were no stripe coat and no caulking, stripe coat with no caulking, stripe coat with caulk on top edge, stripe coat with caulking on top and vertical crevices, and stripe coat with caulking on all sides. The coating system used was a three coat system. The coating sequence used was as follows:

1. Zinc primer
2. Zinc primer stripe
3. Intermediate coat
4. Intermediate coat stripe
5. Caulk application
6. Finish coat stripe
7. Finish coat

The specimens were subjected to accelerated corrosion testing and the performance was judged based on the pit depth on the surface of the corroded region.

Results showed that for new steel specimens the stripe coating with the bottom crevice un-caulked experienced the least amount of corrosion and minimum pit depths. Un-caulked bottom crevice allows moisture to drain down. Priming the surfaces before assembly performed better than priming done after assembly. For weathered steel specimens, the stripe coating with caulking on all sides proved to be the best practice (Shoyer et al., 2018).

4. PACK RUST IN INDIANA BRIDGES

4.1 Procedure Utilized for Study

Consistent with federal requirements, each bridge in Indiana is inspected every two years, and the inspection reports are uploaded to the BIAS (Bridge Inspection Application System) database. The inspection reports after 2006 are digital copies, and those before 2006 were hard copies which were digitized and uploaded on to the database. Using the BIAS database, inspection reports after 2006 of all the state-owned bridges were reviewed in search of pack rust of any form. In some cases, historical reports were also reviewed which were dated back to 1999.

Two methods identified pack rust in steel bridges: first, if the bridge inspector had identified that there is pack rust and had mentioned it in the inspection report, and second by visual observation from the images present in the inspection report.

The number of bridges with pack rust was counted to find how many bridges in Indiana are affected by it. This evaluation allowed the research team to determine how frequently pack rust occurs in Indiana bridges, and to provide some evidence on whether or not pack rust is a problem in the state of Indiana. A dataset of bridges with pack rust was created in Excel. The parameters from the inspection reports which were used to create the dataset included the following information:

- District
- Facility Carried
- Asset Name
- Year Built
- Year Painted
- Member Affected
- Superstructure Rating
- County
- Feature Intersected
- Type of Bridge
- Year Reconstructed
- Pack rust mentioned in the report (Y/N)
- Rating of the member affected
- Year pack rust was first observed

For the bridges with pack rust in splices, latitude and longitude were also noted. These data were then used to find any trends in pack rust occurrence and any parameters which promote pack rust formation.

4.2 Initial Observations

There are seven bridge components or members which were observed to have been affected by pack rust. These components include the following:

1. end diaphragms
2. gusset plates and connections
3. beam cover plates
4. cross bracings
5. hinge-pin connections
6. splice plates
7. bearings (rocker bearings and elastomeric bearings)

The identification of pack rust from images of end diaphragms and bearings was a challenging task. There were many cases of full surface corrosion on the bearings and end diaphragms, which made it challenging to identify pack rust between the contact surfaces. The position and the distance from where the photographs were taken and low-resolution images also made it difficult to identify pack rust. Close-up images of the bridge were helpful in identifying the pack rust more easily. Rust bleeding from the crevice, which indicates the initial phases of crevice corrosion, often could be detected if close-up images were present.

The process of detecting pact rust in bridges has many variabilities since it is dependent on the photographs presented in the inspection report. Some districts may have a practice of taking a greater number of photographs of the bridge than other districts, capturing many minute details. Moreover, each member has locations which are critical to pack rust formation.

4.2.1 End Diaphragms

The overall condition of the end diaphragms in steel bridges of Indiana seems fair. Many of the end diaphragms show surface rusting, and rusting of the top edge. The surface of diaphragms is generally covered by rust bleeding from the top edge, circled as 1 in Figure 4.1.

Figure 4.1 is the most common camera position from which the photos are taken. From this position, the crevice between the connections (location 2 in Figure 4.1) is not visible, and hence minor pack rust cannot be detected. There is a high possibility of pack rust in the crevice at the back of the diaphragms from the water and salt seeping down the deck joint but only rust bleeding can be seen, as shown in location 3 in Figure 4.1.

Figure 4.2 shows pack rust in bridge number (I465) 31-49-04449 B. Pack rust is evident between beam web and the leg of the angle member used to connect the diaphragm. This is the most severe condition of pack rust found in a diaphragm in Indiana. Paint has been applied over pack rust without removing and cleaning it. Taking a closer look at the pack rust, distinctive layers of rust built up over the years can be observed. The bridge was built in 1962 and last painted in 1993. Pack rust was first reported in 1998 and the image presented was taken in 2014. The images for the progression of pack rust are not available. It is difficult to judge if the pack rust thickness is large near the bolts. If the

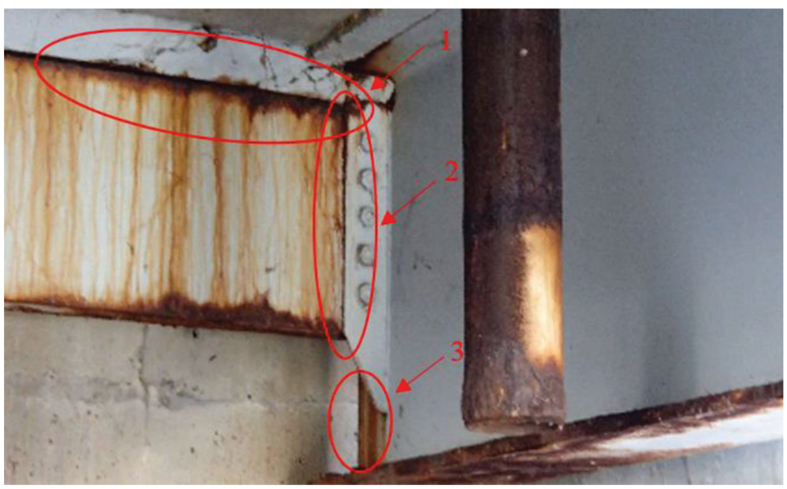


Figure 4.1 Typical camera angle for photos of end diaphragms.

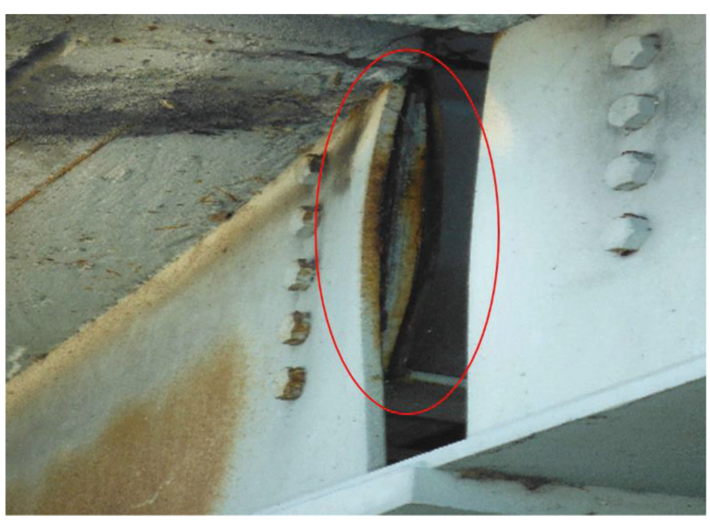


Figure 4.2 Most severe pack rust in the diaphragm found in Indiana.

thickness of pack rust at the line of bolts is roughly 1 inch, the bolts will have enormous additional stress in them.

The expectation of pack rust occurrence in diaphragms was high because of the presence of deck joints above them, which are the entry point for the salt and water. At the end of the inspection process, the expectations were not met. The explanation for the lower number of occurrence is likely due to the limitation set by the images.

4.2.2 Gusset Plate and Other Connections

Pack rust in the gusset plates is seen in all the truss bridges. Most of the truss bridges in Indiana are relatively old, with nearly all built before the 1960s. The most common location where pack rust is observed are the connections in the lower cord of the truss. Figure 4.3 shows a missing rivet in the connection of the bottom chord of a truss bridge. There could be three likely scenarios for the failure of the rivet: first, the stresses in the rivet due to pack rust exceeded fracture strength of the rivet, or second the crevice corrosion led to significant section loss in the rivet causing it to fail, or third a combination of the first two. It is difficult to tell by looking at the image as to what could be the reason for the failure.

Figure 4.4 shows the bottom chord of the truss. The members visible are the angle members connected back to back with a small gap in between (marked as 1). There are two locations in the image where pack rust is present. The pack rust has filled the gap between the members over the years, and there is severe section loss at location 2 circled in the image. The second location is the connection between the gusset plate and the angle members is also affected by the pack rust marked as 3.

Figure 4.3 Rivet failed due to pack rust.

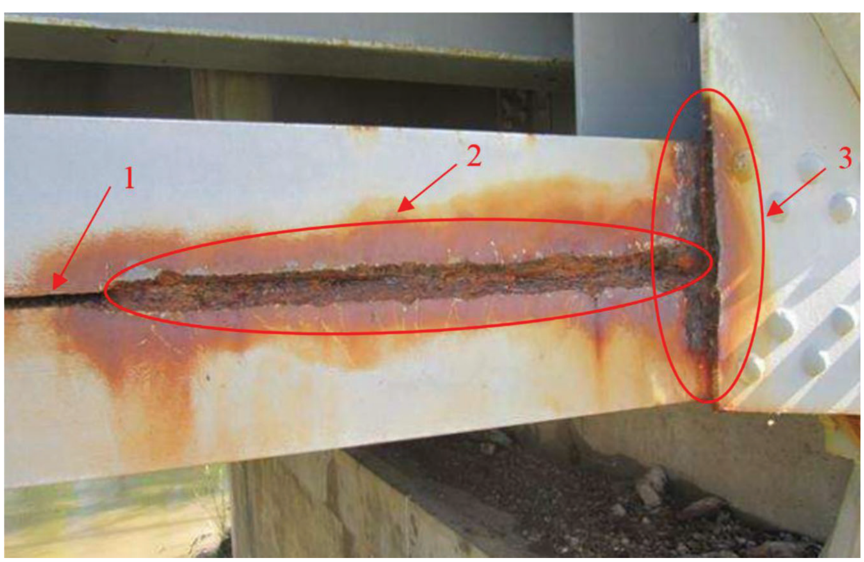


Figure 4.4 Pack rust in the lower chord and gusset plate.

Figure 4.5 Pack rust in connections of the bottom chord.

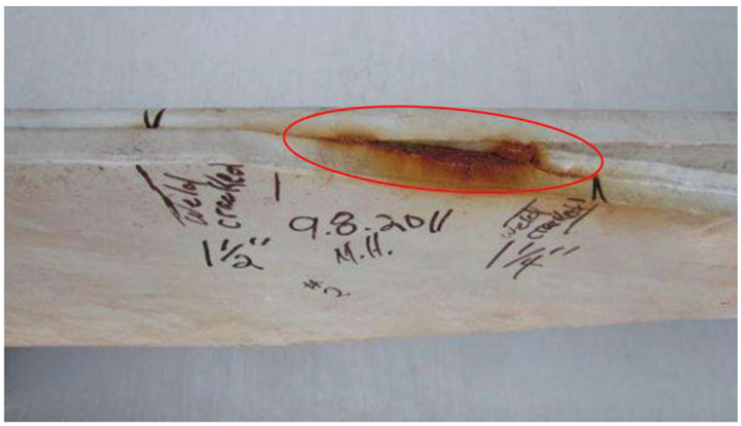


Figure 4.6 Pack rust at the tapered location for wider cover plates (location 1).

Figure 4.5 shows same connection detail after four years. The image on the left was taken in 2013, and the connection had severe pack rust causing section loss and bending of plate. The bridge was painted in 2016. The image on the right is of the same connection in 2017. From the post painting image it looks like some of the rust product has been removed but not completely. It is seen that just after one year of painting, rust bleeding is visible at the locations 1, 2 and 3 marked in the right image in Figure 4.5. There are chances that this is not an active corrosion it could be just rust coloration from the already present rust due to rain. No field testing was conducted for this study.

4.2.3 Beam Cover Plates

There are three specific locations in a cover plate detail where pack rust can occur and those are explained in the following three paragraphs. The cover plates have a tapered design at the ends, and the gap in the welding generally makes the connection susceptible to pack rust formation between the cover plate and the bottom flange.

The first case is if the cover plate is wider than the bottom flange the welding is discontinuous between the straight edge and the tapered edge of the cover plate as shown in Figure 4.6. The portion which remains un-welded serves as an entry point for the moisture into the crevice between the cover plate and the bottom flange. The formation of pack rust in the crevice causes adjacent welds to crack. Figure 4.6 shows bridge inspector's markings for the weld cracks on the bridge to check crack propagation in the future.

The second location in cover plate detail where pack rust occurs is where the welds terminate at the ends of the cover plate. The welds connecting the cover plate and the bottom flange generally terminates at the end and are not continuous, thus leaving an un-welded segment at the end. An example is shown in Figure 4.7. This un-welded part is an entry point of the moisture and initiates pack rust formation. The growth of pack rust in this portion causes welds to crack. Due to the low level of lighting available when the photograph was taken the welds at the end are not clearly visible in Figure 4.8. Figure 4.8 shows a severe case of pack rust at the ends of the cover plate. The welds have cracked, and the cover plates have bent.

The third case in which the pack rust is formed in the cover plates is when the welds are not continuously made as shown in Figure 4.9. The causes for the pack rust formation are the same, non-continuous welds give an entry point to moisture. If the bridges intersect roads, the salt spray created by the moving vehicles gets deposited inside the crevice. Figure 4.9 shows bowing of the cover plate due to pack rust and failure of the welds.

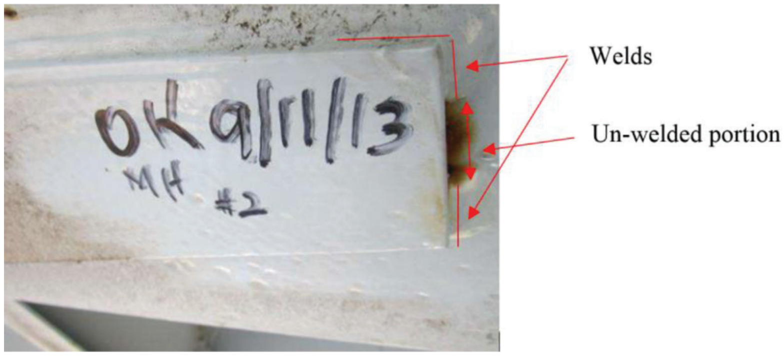


Figure 4.7 Un-welded portion at the end of cover plate (location 2).

Figure 4.8 Pack rust at the end of the cover plate (location 2).

Figure 4.9 Non-continuous welds between the cover plate and bottom flange (location 3).

4.2.4 Cross Bracings

The connection of bracing members with the gusset plate is generally made by welding. Welding does not leave an open crevice to form pack rust, but not all the edges are welded and hence there are edges which leave crevices open. The deck protects the cross bracing from direct contact with water. The only exposure to water is the atmospheric moisture and the salt mist created by the moving vehicles. The limited exposure to the water decreases the probability of the pack rust formation in cross bracings. Not many bridges were observed to have pack rust in cross bracings.

Figure 4.10 shows moderate corrosion between the cross-bracing members and the connection plate. The connection plate seems to have bent at the end due to

Figure 4.10 Example of pack rust in cross bracing.

the development of pack rust. A case was observed where the whole angle section came off and fell on the pier, as shown in the Figure 4.11. The welds undoubtedly cracked due to pack rust.

4.2.5 Hinge-pin Connection

Hinge-pin connection details were observed to be highly prone to pack rust. Hinge-pin connections are located below the deck expansion joints as seen in Figure 4.12 and Figure 4.13 (*left*). The water and salt from the deck flows down into the crevice formed between the hinge plate and the web of the girder. The formation of pack rust prevents the free rotation at the connection. Visual inspection does not give a clear picture of the extent to which the pins have rusted. Ultrasonic tests are required to find the amount of

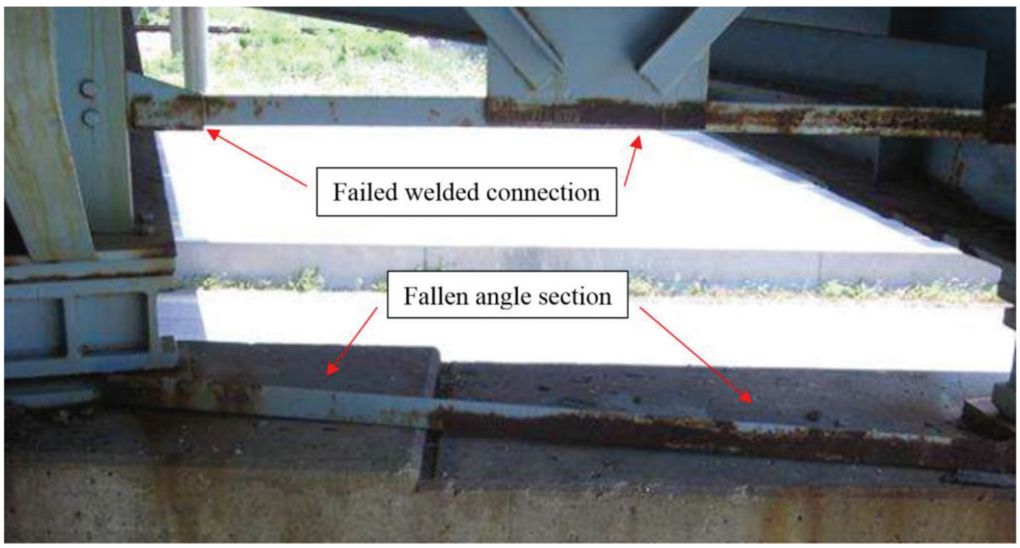


Figure 4.11 Angle connection failed due to pack rust.

Figure 4.12 Finger joint.

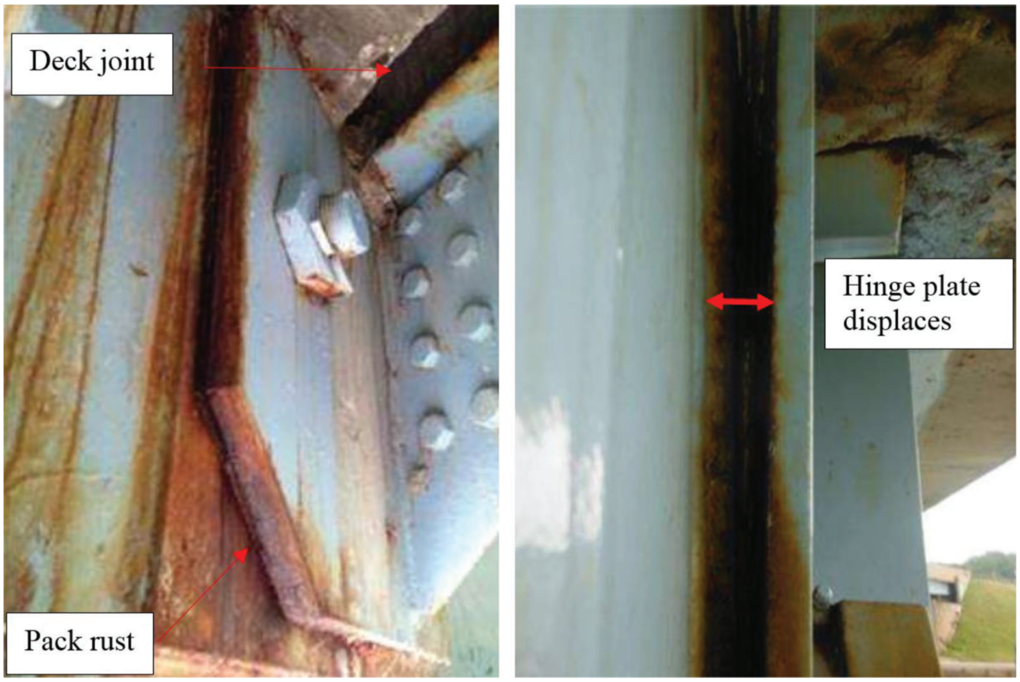


Figure 4.13 Pack rust in the hinge-pin connection.

section loss in pins. Depending on the severity of the section loss of the pin, a severe decrease in the shear capacity could occur. Pack rust between the hinge plates and the girder web causes an out of plane bending of the hinge plate, and it also displaces the hinge plates as seen on the right of Figure 4.13. The out of plane bending will introduce eccentric loading onto the pins for which they are not designed. It was suspected that this was the reason for the collapse of the Minus River Bridge.

Figure 4.12 shows a particular type of expansion joint called finger joint. This kind of joint allows all the material such as debris, water, and salt to pass through it. The seepage of water and salt from the deck to the beam or girder allows for an aggressive crevice corrosion. Hence, this type of joint should not be used, and typically is not, above a pin connection. From Figure 4.13 on the left it can be observed that significant corrosion is occurring between the hinge plate and the web. Figure 4.13 on the right shows an example where the hinge plate has displaced due to pack rust. It is not clear as to the depth of the pack rust and the condition of the pins.

4.2.6 Splice Connection

Splices are present in all the multi-span beam and girder bridges. There are three locations in splice details where pack rust was observed. The first location is at a gap which exists between the bottom flanges and is the most common spot where pack rust in splice connections is observed as shown in the Figure 4.14. This gap is the entry point for the moisture and the salt. Given time, rust starts to pack between the flange plates and the splice plates. With the growth of pack rust the plate starts to bend and increase the stress in the adjacent bolts. If the splice plates bend an excessive amount they may cause large strains in the bolts, which may result in fracture. Deformation of the splice plate depend on the thickness of the splice plate, amount of pre-tensioning in bolts, and the edge distance of the bolts.

The other two locations where pack rust is observed are the corners and the edges of the splices, as shown the Figure 4.15 and Figure 4.16 respectively. The occurrence of pack rust in these two locations was observed in few cases.

Figure 4.15 shows pack rust at the corner of the splice. There are two things to observe from the image: first, the bolts are arranged in a staggered pattern, and second, the splice plate is not very thick. The staggered pattern of bolts used in this case increases the distance between the nearest bolt and the corner. This causes a reduction in the clamping force at the corner. Reduced clamping force will cause a larger crevice height and thus allow easier entry of salt and water. If the arrangement of the staggered bolt pattern used were just mirror image, then the closest bolt would be at a distance nearer to the corner compared to the current case.

The chances of bolt fracture are less if the plate is thin. This is primarily because the forces exerted by pack rust deposit can easily bend the plate if it is thinner. Because the splice plate can easily bend, it would predominately bend rather than having a rigid body movement which would be the case in thick plates. This theory holds true only if the growth of pack rust is limited towards corners. When pack rust starts to grow inwards, it would eventually increase stresses in the bolts.

Figure 4.16 shows a riveted built-up bridge girder. It has a staggered rivet arrangement. It is evident that at

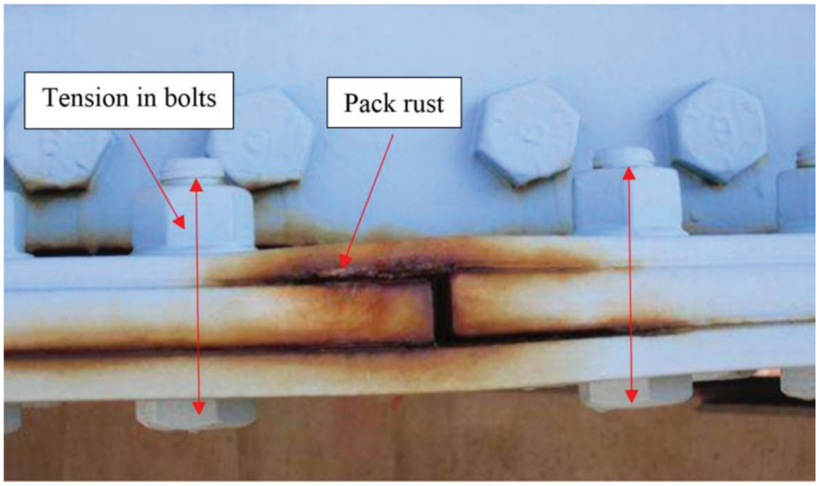


Figure 4.14 Pack rust in the middle of the splice plate (location 1).

Figure 4.15 Pack rust in the corners of the splice plate (location 2).

Figure 4.16 Pack rust in the edges of the splices (location 3).

the locations where the rivet is positioned away from the edge, pack rust and rust bleeding is visible at those locations only. Figure 4.16 sets a clear example that edge distance does play a major role in preventing pack rust. Also, the clamping action provided by rivets is much less than that provided by high strength bolts.

4.2.7 Bearings

The rocker bearings are notably affected by pack rust. Only a very few cases were observed where elastomeric bearings were also affected by pack rust. In many cases, there was surface corrosion on the bearings which made it difficult to identify and judge whether pack rust is present or not. Bearings often are in contact with water and salts, as deck joints are often present where bearings are placed.

Figure 4.17 shows pack rust between the rocker bearings and the masonry plate. Figure 4.18 shows pack rust between the retainers and the sole plate. There are many cases where multiple shim plates are inserted between the sole plate and the beam/girder to adjust the height and make a fit. This leads to the formation of multiple crevices between shim plates and results in the formation of pack rust in these crevices.

Even though pack rust in rocker bearings occurs quite frequently, mitigating pack rust in rocker bearings is not a high priority since INDOT plans to eventually reconstruct all the rocker bearings with semi-integral bearings, which are not prone to pack rust formation.

Figure 4.17 Pack rust in the rocker bearings.

4.3 Statistics and Analyses

Statistics of the number of bridges that have pack rust and number of bridges with pack rust in a particular member are presented. This helped in answering the question if pack rust occurs frequently in the steel bridges in the state of Indiana and which members are most affected by pack rust. Analyses were performed on the data collected. Analyses involved identifying possible parameters which could influence pack rust formation. The identified parameters that could have an influence on pack rust formation were based on location north to south and salt usage, and features intersected by the bridge (water, roads, railroads, and abandoned railroads). These two parameters were used on all the bridges that have pack rust. Each member type would have different influencing parameters. Depending on member type further parameters were identified.

The total number of steel bridges in Indiana is 1,781, of which 571 bridges have some form of pack rust in at least one component of a bridge, such as splices, bearings, connections, and other members as mentioned earlier. A bridge can have pack rust in multiple components. For example, a bridge may have pack rust in its bearings, splices, and beam cover plates, but it will be counted only once. About a third of the steel bridges in Indiana has pack rust that ranges from minor pack rust where rust has just started to form to very severe pack rust where welds have cracked or bolts have fractured.

In all the graphs and tables presented, the districts are arranged in the order of their locations from districts in the north to the districts in the south of Indiana. Districts are divided into three groups: north includes Fort Wayne and LaPorte, middle includes Crawfordsville and Greenfield, and south includes Seymour and Vincennes.

For the Fort Wayne, Crawfordsville, and Greenfield Districts, the number of bridges with pack rust is fairly

Figure 4.18 Pack rust in elastomeric bearings.

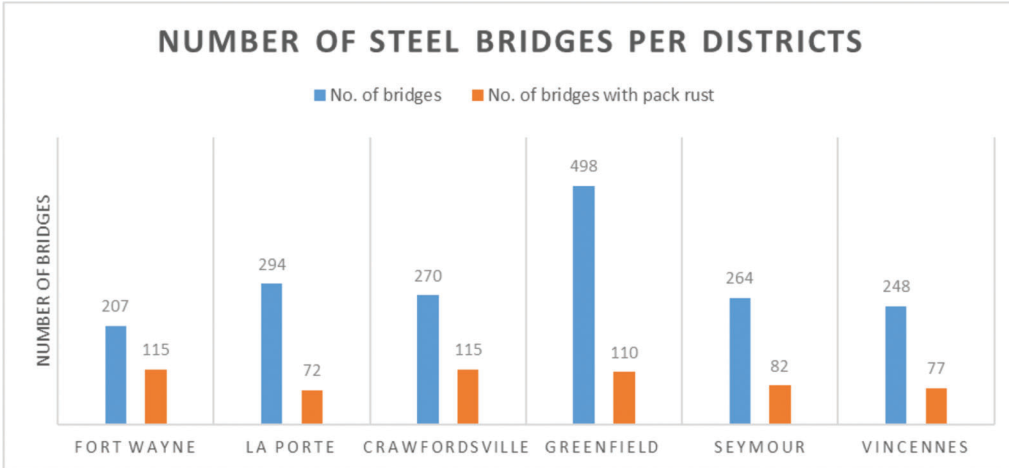


Figure 4.19 Total number of bridges and bridges with pack rust in different districts.

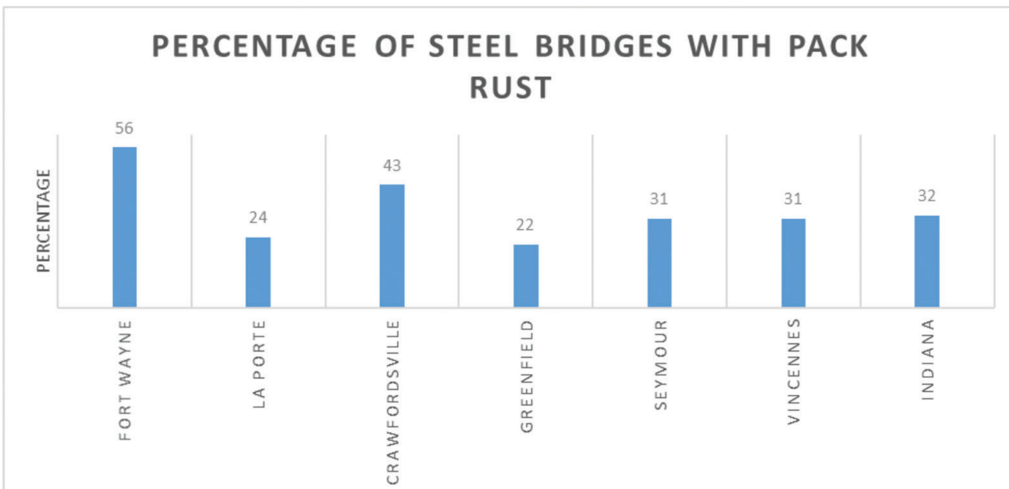


Figure 4.20 Percentage of bridges with pack rust in districts and Indiana.

close; moreover, the number of bridges with pack rust is fairly close for the LaPorte, Seymour, and Vincennes Districts (refer to Figure 4.19). It is also essential to compare the percentage of bridges in a district that have pack rust. Comparing the percentage of bridges having pack rust in Figure 4.20, Fort Wayne has the highest percentage, with more than half of the bridges having some form of pack rust. On the other hand, 22% of bridges in Greenfield have pack rust, which is least among all the districts in Indiana.

Figure 4.21 shows average age of the steel bridges in each district. The relative age of bridges in each district does not differ by much from the average of steel bridges in Indiana, which is 48 years. The average age of the steel bridges is highest in Crawfordsville district but, the pack rust percentage is not highest. The average age of bridges in Seymour is higher than average of bridges in Fort Wayne by 4 years but, the pack rust percentage in Fort Wayne is almost twice of that in Seymour. No distinct correlation seems to exist between average ages of the steel bridges and pack rust percentage, because the average age of the steel bridges in each district are in close range. This does not mean, however, that pack rust occurrence is not dependent on the age of the bridge. Generally, problems with pack rust would increase with the increasing age of a bridge. However, it would obviously also depend on the maintenance practices on a bridge over its life time.

4.3.1 Location, Salt and Brine Usage

Initial expectations were that the districts in the north would have a higher percentage of pack rust than the districts in the south. The pack rust percentage in LaPorte and Greenfield do not seem to follow the expected trend. The expectations were based on the amount of average annual snowfall shown in Table 4.1, which decreases from the north to the south.

Table 4.1 lists the district-wise breakdown of the average snowfall data in column 2 (U.S. Climate Data, n.d.), salt usage per lane miles and brine usage in columns 3 and 4 (INDOT, 2018). This table provides

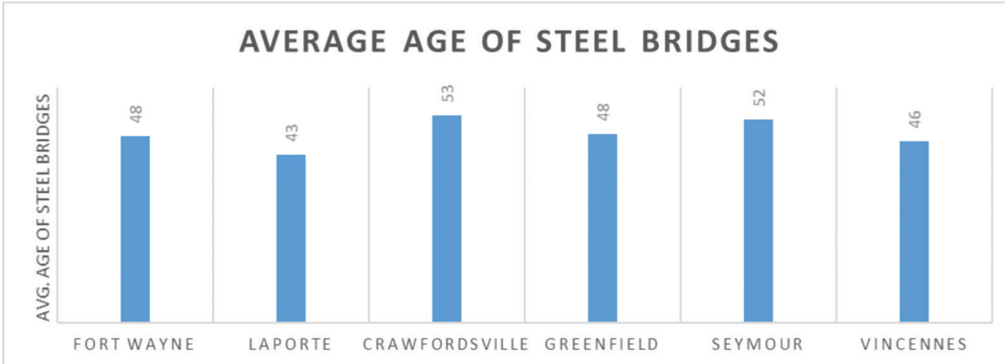


Figure 4.21 Average age of steel bridges in each district.

TABLE 4.1
District wise breakdown of average annual snowfall, salt usage, brine usage, and pack rust percentage

District	Avg. annual snowfall (in.)	Salt usage tons/lane-miles (5 yrs. avg.)	Brine used (gal.) (5 yrs. avg.)	Pack rust percentage
Fort Wayne	34	11.1	191,673	56
LaPorte	61	13.2	2,807,355	24
Crawfordsville	20	9.5	170,844	43
Greenfield	20	10.2	332,312	22
Seymour	8	7.5	583,539	31
Vincennes	9	4.1	121,634	31

information that was used to identify possible correlations between the salt usage and the pack rust percentage from north to south.

The average annual snowfall (U.S. Climate Data, n.d.) in LaPorte is the highest and is almost seven times more than that reported in southern Indiana. Consequently, the salt and brine usage in the northern districts is more than that in the southern districts. The snowfall reported in the two northern districts have a large difference due to the lake effect snowfall that occurs in the LaPorte District. Although the snowfall in LaPorte is twice of that in Fort Wayne, the salt usage in LaPorte is only 15% more. The lower salt usage given the snowfall totals in LaPorte and Fort Wayne is compensated by higher brine usage in the LaPorte. However, the pack rust percentage in LaPorte is only half of that in Fort Wayne. Crawfordsville and Greenfield experienced nearly the same amount of snowfall, and the salt usage is almost equal, but Greenfield uses twice the amount of brine used by Crawfordsville. However, the percentage of bridges that have pack rust in Greenfield is half of that observed in Crawfordsville. Therefore, the low percentage of pack rust in these two districts, LaPorte and Greenfield, are not the result of the low salt usage, in fact, the salt usage is more. In the southern districts, there is also a discrepancy in the salt usage and the pack rust percentage. The salt and brine usage (tons/lane-miles) are less in the Vincennes compared to that in Seymour, but the pack rust percentage in both the districts are observed to be the same. No factors were identified for this observation.

The lower pack rust percentage observed in the LaPorte and the Greenfield Districts may be the result of a good maintenance practice used in both the districts. Discussion with the Study Advisory Committee members confirmed that both of these districts tend to be more proactive in regards to annual pressure washing of the bridge bearings and superstructure. It is hypothesized that this regular cleaning of the structure to remove debris and salts may have delayed the formation of pack rust and led to the lower pack rust frequency of occurrence observed.

Figure 4.22 shows district wise breakdown of salt usage in tons/lane-miles vs pack rust percentage. Except for LaPorte and Greenfield other four districts should show some correlation between salt usages (tons/lane-miles) and pack rust percentage. The districts Fort Wayne, Crawfordsville and Seymour seems to follow a trend of decreasing pack rust percentage with decreasing salt usage (tons/lane-miles). Vincennes does not seem to follow the same trend and experiences higher pack rust percentage than expected. Based on the best-fit line using data points of Fort Wayne, Crawfordsville and Seymour, Vincennes is expected to show a pack rust percentage near about 10%.

4.3.2 Feature Intersected

The steel bridges are built over various features, and it was examined whether or not the type of the feature passing below the bridges may influence pack rust formation on the bridge. In Indiana, the bridges intersect features including water bodies like rivers, creeks, ditches, and forks; roads including interstates, US highways, State roads, streets, and county roads; and railroads and abandoned railroads. All bridges are

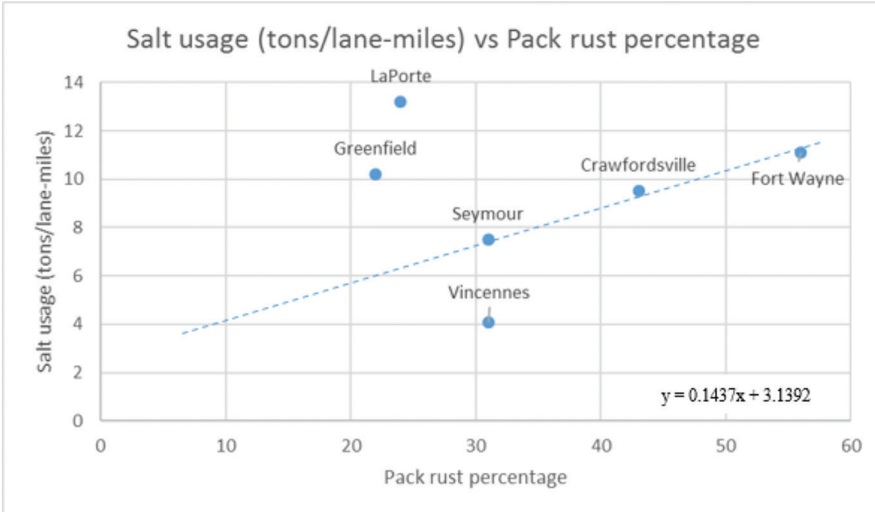


Figure 4.22 District wise breakdown of the salt usage (tons/lane-miles) vs pack rust percentage.

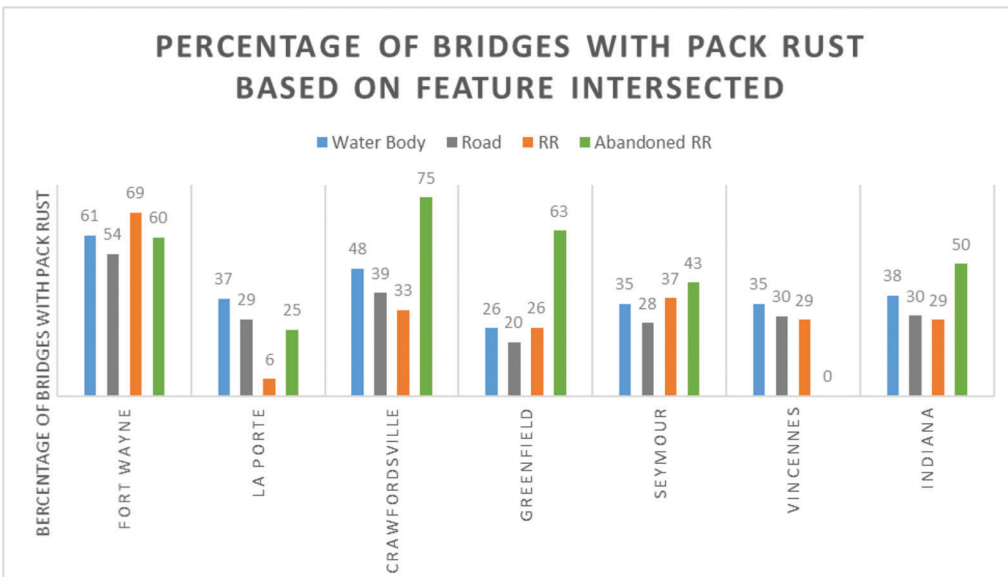


Figure 4.23 Percentage of bridges with pack rust based on features intersected.

segregated into only four groups, which include water bodies, roads, railroads, and abandoned railroads. This is done to find if any particular feature that intersects a bridge has a significant influence on pack rust formation or the time it took to form pack rust. The moisture content in the atmosphere near the bridge that intersects water bodies will likely be greater than the bridge that intersects other features. The salt accumulation on bridges that intersects roads will likely be greater due to misting of the salts present on the roads by moving vehicles. The soot deposit from the rail engines underneath the bridges that intersect railroads is expected to have different corrosion chemistry. The bridge that intersects abandoned railroads are generally well vegetated, and because of good vegetation, the moisture in the region will likely be high and also there will be deposits of soot.

In Figure 4.23, the percentage of the bridges with pack rust that intersect abandoned railroads stand out very prominently for the Crawfordsville and Greenfield Districts, but the number of bridges is a small number totaling to 22 bridges out of 44 bridges in Indiana. The number of bridges that have pack rust and the total number of bridges that intersects specific feature is presented in Table A.2 in the Appendix.

From the percentage perspective, abandoned railroad appear to be the most concerning feature intersected for its high percentage values in four districts, but the overall number of bridges are small. In general, the trends in individual districts are not distinctly clear and consistent. The percentages are highest for the bridges that intersect water bodies (after abandoned railroads) except for Fort Wayne, where the highest percentage is observed for railroads. The percentage of pack rust in

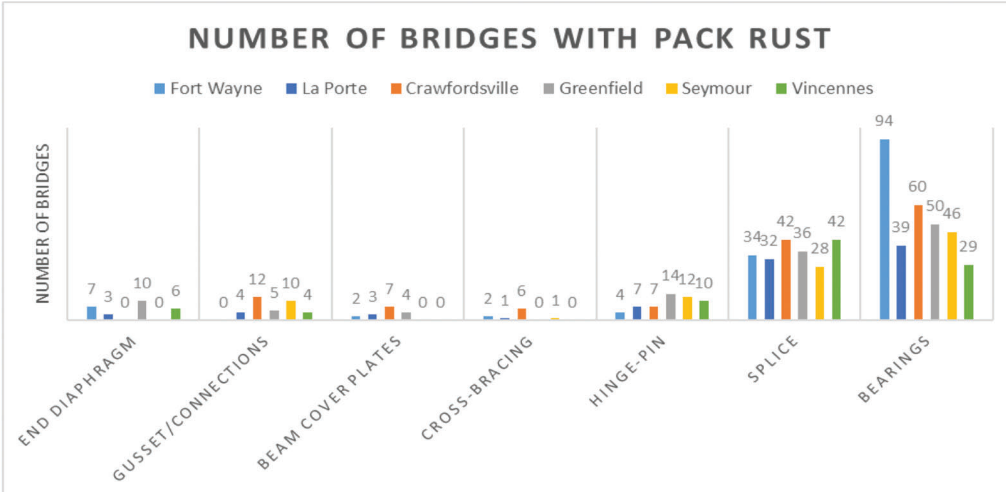


Figure 4.24 Number of bridges with pack rust in bridge members.

the bridges that intersect railroads are higher than the bridges that intersect roads in the eastern districts of Indiana, i.e., Fort Wayne, Greenfield, and Seymour, while the reverse is true in the western districts of Indiana, i.e., LaPorte, Crawfordsville, and Vincennes. No reason was found as to why this particular trend exists. In total, the percentage of pack rust in bridges that intersects roads and railroads are almost same. However, the number of bridges that have pack rust and intersects roads are three times the number of bridges that have pack rust and intersects railroads.

The influence of the feature being intersected by the bridge on pack rust in bearings, hinge-pin connection, and end diaphragms is expected to be overshadowed by the joint condition. Therefore, the effect of the intersecting feature should be studied for each member.

4.3.3 Member-wise Breakdown of the Number of Bridges That Have Pack Rust

It is important to segregate the data based on the component or member of the bridge that is experiencing pack rust because the factors associated with causing the pack rust in each component will be different. For example, the probability of pack rust formation in rocker bearings will be substantially dependent on the joint condition as a bad joint condition will allow water and salt to flow down to bearings. Joint condition does not typically influence pack rust formation in splice connections.

Figure 4.24 illustrates the breakdown of the number of bridges with pack rust in a particular member of a bridge in each district. From these data, it is observed that the number of bridges that have pack rust in splice details and bearings is large. Pack rust in the other members does not seem to be as common. The number of bridges in LaPorte that have pack rust in bearings is less than half of that in Fort Wayne. As noted before, it is believed that this is because LaPorte has a dedicated maintenance crew that annually pressure washes the bearings from all the dirt and salts using water jets.

The number of bridges that have pack rust in a specific member of a bridge does not give an overall picture of how frequently pack rust occurs. Looking at the percentage of occurrence is important. Numbers for the end diaphragm and cross bracings are less in Figure 4.24, and so are the percentage of bridges that have pack rust. A number of bridges were observed with general corrosion problem in end diaphragms, but it is difficult to identify if pack rust exists between the diaphragm and the connection plate, so only few bridges were identified to have pack rust in diaphragms from images and it is likely that many bridges with pack rust in end diaphragms went unidentified.

Pack rust in beam cover plates is not very common. In Indiana, only 16 bridges have pack rust in cover plates. However, the total number of bridges in Indiana that have cover plates is tedious to find, so the probability of occurrence of pack rust in cover plates is not calculated for the entire inventory of steel bridges in Indiana. For the Crawfordsville district, a total number of bridges with cover plate details is 58. Therefore, the occurrence of pack rust in Crawfordsville district in cover plates is about 12%.

Gusset plates and other connections details are commonly used in truss bridge construction. Moreover, it has been observed that nearly all of the truss bridges in Indiana have pack rust in some of the gusset plates, but not all the gusset plates in a given bridge have pack rust. The most common location in truss bridges where pack rust occurs is at the connections in the bottom chord. The washed away salt from the deck is one of the primary factors causing aggressive pack rust in the bottom chord of a truss bridge.

The total number of bridges with pack rust in hinge-pin connections is small, but almost all the bridges with this connection detail have pack rust. The percentage in Crawfordsville is 100%. The reason for this observation can directly be correlated with the fact that there are often deck joints present directly above the hinge-pin connections, which allows water and salt to spill down onto this connection.

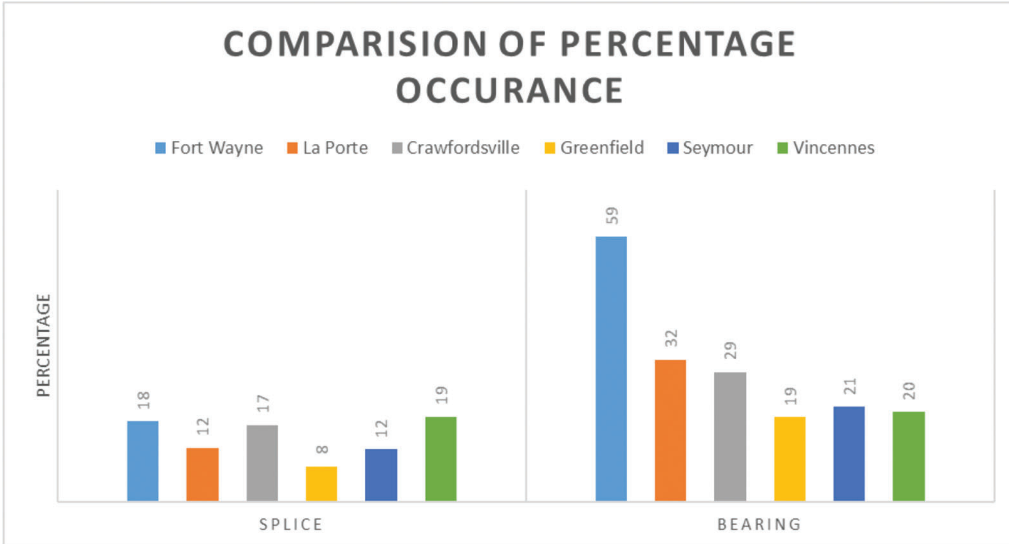


Figure 4.25 Percentage of pack rust occurrence in splices and bearings for each district.

The total number of bridges with pack rust in splices and bearings stands out from the rest of the details. Figure 4.25 shows the distribution of the pack rust percentage for the splices and bearings for each district.

The percentage of pack rust occurrence in a splice in the districts from the north to the south does not appear to follow any trend based on the average snowfall, salt, and brine usage. The Vincennes District, which experiences nearly the least amount of snowfall and uses the least amount of salt and brine, has the highest percentage of pack rust in splices. The LaPorte District, on the other hand, has highest average snowfall and brine usage (refer to Table 4.1). The Greenfield District, meanwhile, has the least percentage of pack rust in splices. It is believed that the lower percentage values could be because of either a lower documentation of pack rust in the inspection report photographs or good maintenance practices in the Greenfield District to pressure wash the bridges, or a combination of the two factors.

The percentage of pack rust occurrence in bearings seems to follow a trend, i.e., higher percentage of pack rust in the northern district and lower in the southern district. The Greenfield district seems to be an outlier in the trend. This lower percentage of pack rust occurrence in bearings was thought to be because of possibly good maintenance of bridges in the Greenfield District. The pack rust in bearings is completely dependent on the condition of the deck joints. Table 4.2 shows that the average NBI rating for joints that have bearings beneath that have pack rust improves by a very small value as northern districts are compared with southern districts.

4.3.4 Pack Rust in Splices

The splice detail is given more importance over bearings because most of the rocker bearings will eventually be replaced by elastomeric bearings which are less prone to pack rust or they will be encased bearings or semi-integral bearings which are not prone to pack rust. Replacement or repairs of the splice affected by pack rust is a costly and challenging job.

TABLE 4.2
Average joint rating for the bridges with pack rust

District	Avg. NBI joint ratings for the bridges with pack rust in bearings
Fort Wayne	4.16
LaPorte	4.2
Crawfordsville	4.22
Greenfield	4.22
Seymour	5.26
Vincennes	4.6

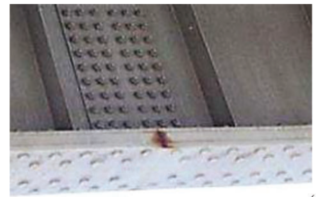
4.3.4.1 Pack rust ratings for splices. A rating system was developed to evaluate and rate the condition of splices affected by pack rust. Ratings from 1 to 5 were given to the splices that have developed pack rust, with 1 being severe and 5 being minor. A detailed description of the rating with examples from Indiana is provided in Table 4.3. The splices that did not have pack rust were not given any rating.

The ratings were given to the splice details for which the images (a total of 177 bridges) were available in the inspection reports. There were 37 bridges where pack rust in the splices was mentioned in the inspection reports, but the images of the splice with pack rust were absent. Figure 4.26 shows the number of bridges that have pack rust in splices together with their corresponding ratings. It is observed that majority of the bridges have a rating of 3 and 4. Rating 5 is given when rust bleeding is observed but, these cases are generally not reported or given a serious concern. It is therefore likely that there could be more bridges with splices of rating 5 that were not recorded and are unaccounted in this study.

TABLE 4.3
Pack rust (PR) severity rating for splices

Rating	Description	Examples
1	Severe PR: >¾ inch bowing of splices or bolt failure	 (a)  (b)  (c)
2	Moderate to severe PR: ¼ inch to ¾ inch bowing of splice plates	 (d)
3	Moderate PR: visible bowing of the splice plates, <¼ inch	 (e)
4	Minor to moderate PR: visible corrosion in middle, edges, or corners of the splice connection	 (f)

TABLE 4.3
(Continued)

Rating	Description	Examples
5	Minor PR: rust bleeding in middle, edges, or corners of the splice connection	 (g)

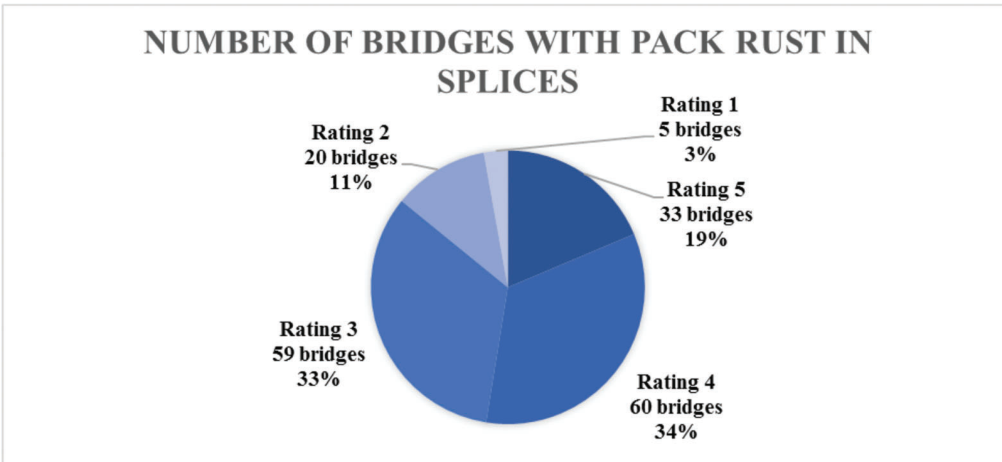


Figure 4.26 Number of bridges with pack rust in splices classified by pack rust rating.

4.3.4.2 Location of splices that have pack rust. The section illustrates which splices are most affected by pack rust. Figure 4.27 shows that the splices that are located on the exterior beams have the highest probability for formation of pack rust in the exterior face of the splice. There are 66 bridges that have pack rust in splices which are on both the exterior beams and interior beams. Only 2 bridge were observed to have pack rust in splices located in interior beams only, with no pack rust observed in exterior splices. About 7 bridges were observed to have pack rust on the inside face of splices on the exterior beams. An example for this can be seen in Figure c and Figure d in Table 4.3. It should be noted that there is a possibility that pack rust is present on the exterior face also, but no images were present in the reports.

4.3.4.3 Effect of features intersected by the bridge on pack rust in splices. Splices are located below the deck for all the beam and girder bridges, so the pack rust in the splices can be significantly influenced by the features present below the bridge. Figure 4.28 illustrates the percentage of bridges with pack rust in splices over the features that are intersected by the bridge for each district and for Indiana overall. It was expected that the percentage would be highest for the bridges that intersect roads because of the salt spray effect created by moving vehicles, but this was not the observation. The general trend observed was that the pack rust percentage is highest for water, then roads followed by railroads. Pack rust in splices over abandoned railroads is found in only 3 districts with no distinct pattern based on geographical location. For Indiana as a whole, the trend remains the same excluding the abandoned railroads. The number of bridges with pack rust and a total number of bridges that intersect a given feature is tabulated in Appendix Table A.3.

The average rating for the bridges which intersect a particular feature is shown in Figure 4.29. The condition of the splices in the bridges over abandoned bridges is better than all the other three features. The severity rating of splices over water bodies is least among all other features intersected. The average rating for the bridges intersecting roads and railroads does not differ by a large value from the average rating of the bridges intersecting the water bodies.

4.3.4.4 The relationship between the number of years after painting and pack rust severity in splices. A correlation between the pack rust severity in splices in the observed year and the number of years after the bridge is painted plotted in Figure 4.30. This is done to

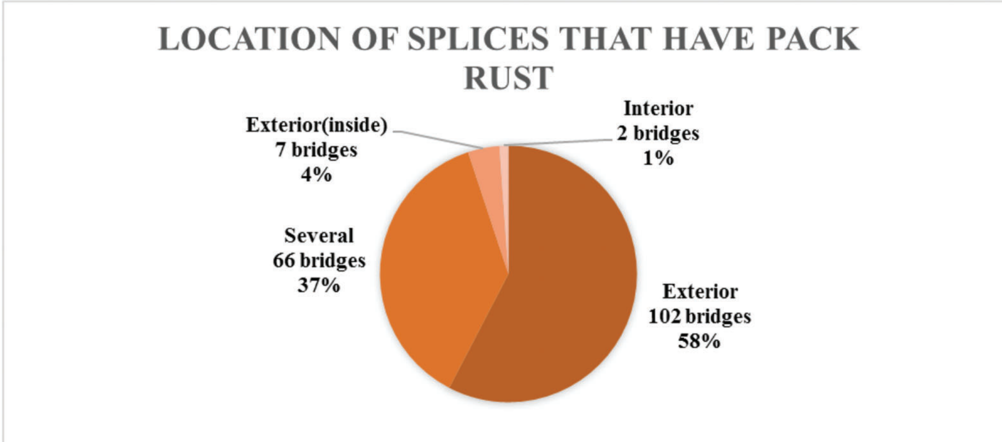


Figure 4.27 Location of splice connections that have pack rust.

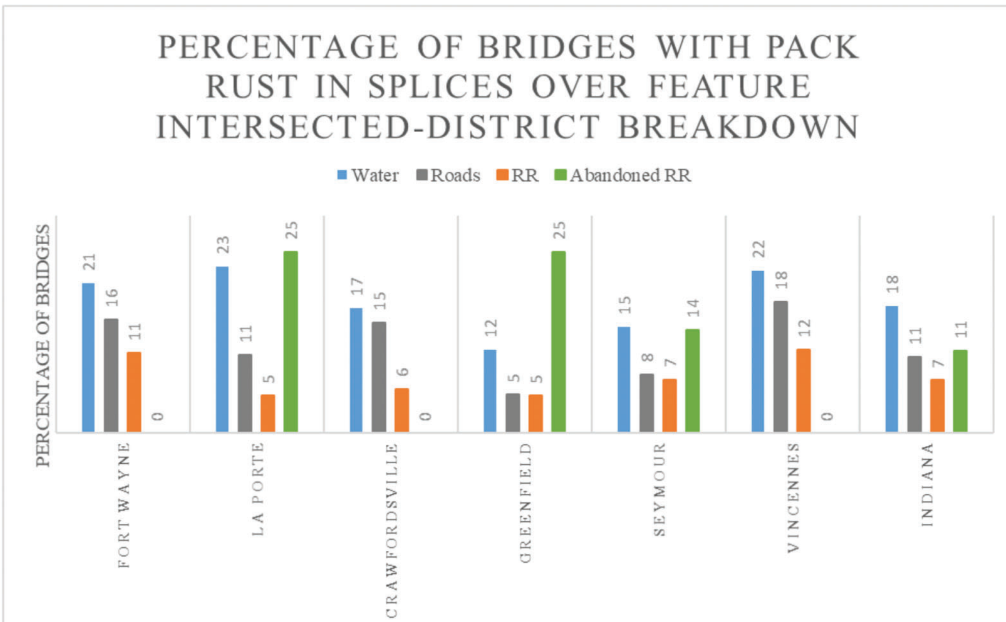


Figure 4.28 Percentage of bridges with pack rust in splice over feature intersected.

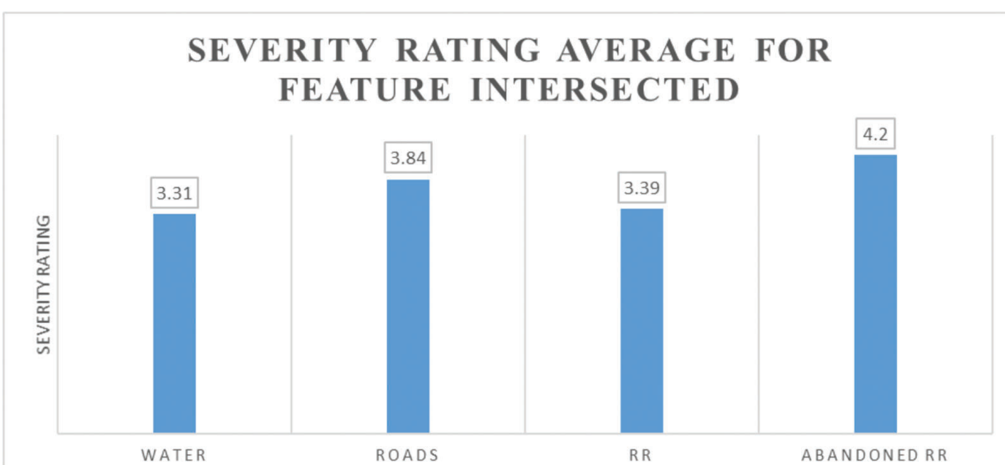


Figure 4.29 Average severity rating for the intersecting feature.

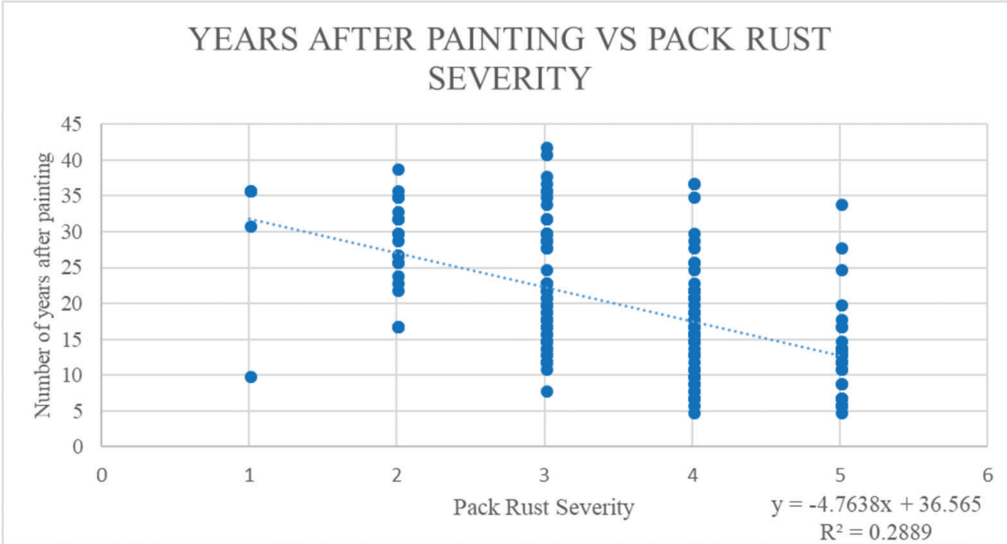


Figure 4.30 Relation between years after painting and pack rust severity in splices.

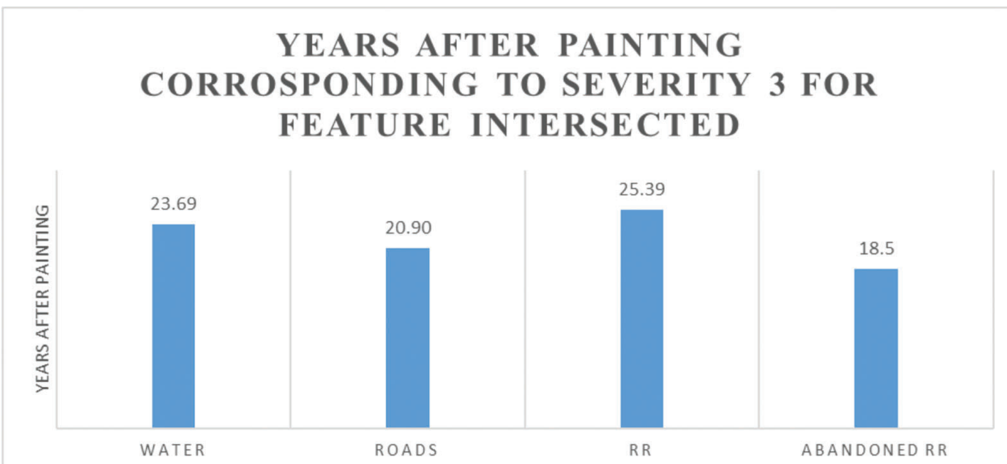


Figure 4.31 Years after painting corresponding to severity rating 3 for feature intersected.

estimate how long it will take for the pack rust in splices to go from minor pack rust to very severe pack rust. The number of years is counted from the year the bridge was most recently painted to the year when splices were observed to exhibit pack rust, and ratings were given. If it is observed that the pack rust was already present in the most recent painting job, then the number of years is counted from the previous painting job.

Based upon the data, a linear best fit line is constructed using the least squares method. From the best fit line, the expected time to form minor pack rust and very severe pack rust is around 12 and 32 years, respectively, after painting (in majority of the cases it is the second or third re-painting of the bridge). This does not imply that all bridges will get minor pack rust 12 years after painting. Since there is only a 13% chance for pack rust to occur in a splice connection. Also, there is considerable scatter in the data (R^2 = 0.29), with some bridges taking more than 25 years after painting to form minor pack rust in splices to one bridge taking just 10 years to reach very severe pack rust.

The trend line equation is used to find which intersecting feature causes the fast formation of pack rust. The number of years after the painting of a bridge is calculated corresponding to pack rust rating of 3 in splices using the slope (m = -4.764) of the best-fit line for each bridge with pack rust in splice connection. For example, if it took 12 years to form a pack rust of rating 5, than the number of years it will take to reach to rating 3 is obtained using the slope of the best-fit line. This is done for every bridge with pack rust in splice connection than separated based on features intersected by the bridge and average number of years to reach to rating 3 is obtained. The Figure 4.31 shows that pack rust of severity 3 in bridges intersecting roads reaches before it reaches in the bridges intersecting

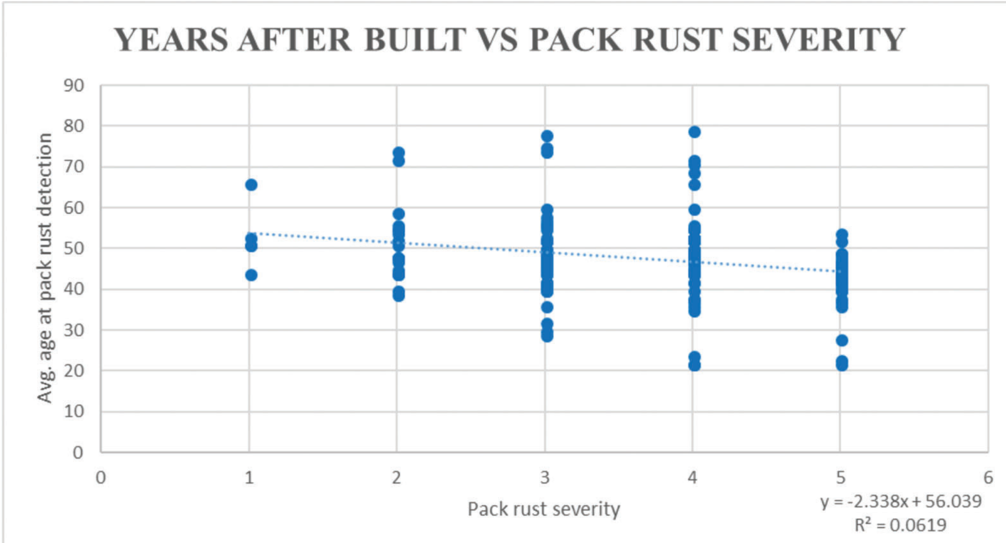


Figure 4.32 Relation between years after built and pack rust severity in splices.

water and railroads. Although the difference is not large the three to four years of acceleration could be because of the use of salt on the roads. The pack rust formation is fastest in abandoned railroads, and no factors were brought to light that can explain the faster rate of corrosion in bridges intersecting abandoned railroads probably due to relatively smaller sample space.

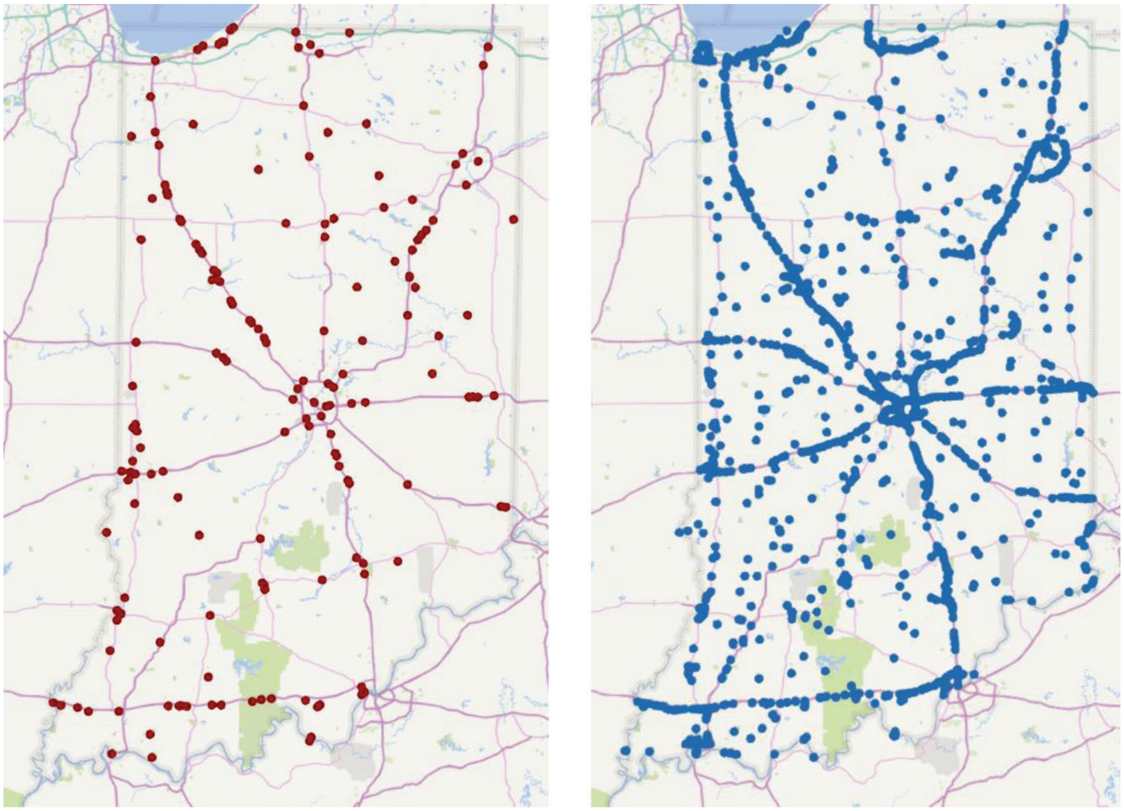
4.3.4.5 The relationship between years after built and pack rust severity in splices. Figure 4.32 plots the data for the severity rating of the pack rust in splices and the age of the bridge at pack rust detection. The best fit line starts from 45 years for minor pack rust to 53 years for severe pack rust. The newest bridge which has pack rust in splices has an age of 21 years. The oldest bridge with pack rust has an age of 79 years but has a moderate to minor pack rust.

4.3.4.6 Vertical under clearance for bridges that intersect roads. The salt spray effect created by moving vehicles below the bridge is believed to be one of the factors causing pack rust in splices. Generally, a greater vertical clearance between the bridge superstructure and the road passing below the bridge will result in less salt deposit at the splices of the bridge. A reduced salt deposit will lead to a slower corrosion rate. Based on this reasoning the parameters involving under clearance, years after painting and the severity of the pack rust were compared to find a correlation. It took 23 years in a bridge that has an under-clearance of 13.5 ft. to reach a condition of rating 3, and 21.5 years for a bridge with an under-clearance of 25.5 ft. (21.5 years is the extrapolated value from the best-fit line Figure 4.30). It is clear from the data in Table 4.4 that the bridges with large under-clearance, the frequency of pack rust occurrence in splice connection is less. No correlation was found between the vertical under clearance, years after painting and pack rust severity.

TABLE 4.4
Number of bridges with pack rust in splice connection with regards to under-clearance height

Under-clearance (ft.)	No. of bridges	Total no. of bridges	Percentage
13–14	1	10	10.0
14–15	10	194	5.2
15–16	20	166	12.0
16–17	31	399	7.8
17–18	2	69	2.9
18–19	2	22	9.1
>19	4	92	4.3

4.3.4.7 Location of bridges that have pack rust in splices. The location of bridges that have pack rust in splices was marked on a map to find if any pattern for pack rust severity and location exists. Figure 4.33 locates all the steel bridges in Indiana with pack rust (of any rating) in splices whose images are available. Figure 4.33 (b) shows location of all the state owned steel bridges in Indiana. This study was conducted particularly for the state owned steel bridges. Figure A.2 shows location of all steel bridges in Indiana. Almost all bridges that are identified to have pack rust in splice connections are on interstates and other NHS routes. These routes represents highest priority/traffic routes for snow removal. There are some stretches of highways where there are no steel bridges that were identified to have pack rust in splice connection. There would be number of steel bridges which are not state owned and have pack rust in splice connection which are not covered in this study. The distribution of bridges that have pack rust in splices based on the rating value is presented in the Appendix Figure A.1.



(a) State owned steel bridges with pack rust in splices (b) All-state owned steel bridges in Indiana

Figure 4.33 Location of all state owned steel bridges and bridges with pack rust in splice connection.

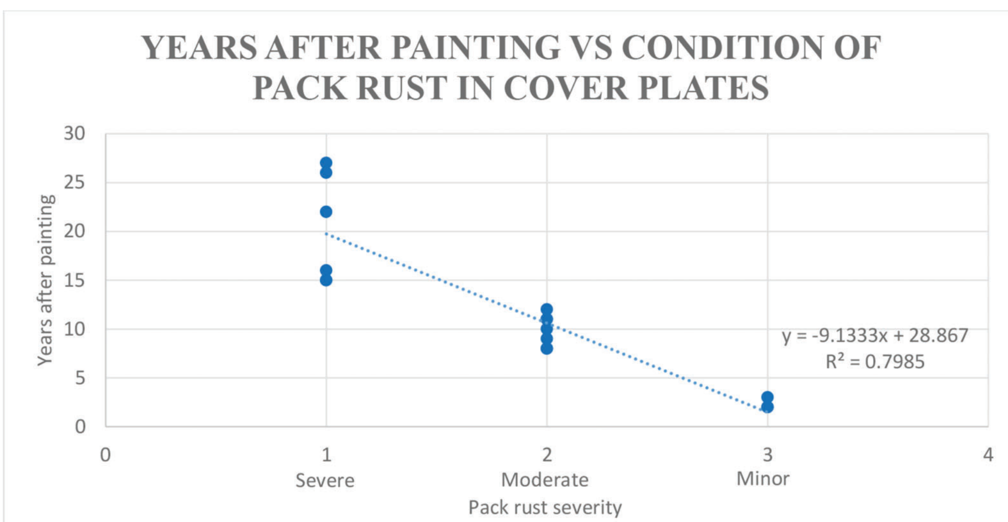


Figure 4.34 Number of years after painting and the condition of pack rust in cover plates.

4.3.5 Pack Rust in Beam Cover Plates

The parameters influencing the formation of pack rust identified for cover plates are:

1. Number of years after painting
2. Feature intersected

4.3.5.1 The relationship between the number of years after painting and pack rust severity in cover plates. Ratings from 1 to 3 were given to the cover plates with pack rust, 1 being severe pack rust where there exist major weld failures and bending of cover plates, rating of 2 where welds have cracked significantly and a rating of 3 where pack rust have just started to form, and rust bleeding is observed. The pack rust ratings were plotted against the number of years after painting the bridge to the year when pack rust was observed. It should be noted that painting here refers to either the initial coating or after one or two repainting; the majority of cases are after repainting. Figure 4.34 illustrates a best-fit line that provides the probable condition of pack rust in cover plates after a certain number of years of painting the bridge. If the bridge was painted 10 years ago it does not mean that the cover plates will have moderate pack rust, but rather that there is only 12% chance for the pack rust to occur in cover plates in Crawfordsville district (refer to section 4.3.3). The time range for severe pack rust to develop is from 15 years to 27 years while the time range for the minor and moderate pack rust is not that wide. The correlation in the data is very good with R^2 of about 80% compared to that for the splices which is less than 30%.

4.3.5.2 Effect of features intersected by the bridge on pack rust in cover plates. The 16 bridges which were identified to have pack rust in the cover plates out of which 15 bridges intersected roads, of which 4 bridges also intersected railroads along with roads and 1 bridge intersected railroad only. The frequency of pack rust occurrence in cover plates of bridges intersecting roads is highest among all the four intersecting features.

5. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

5.1 Summary

The study found that about one third of the steel bridges in Indiana have developed pack rust to some degree. In general, the percentage of bridges that have pack rust decreases from the districts in the north to the districts in the south. However, two of the districts were found to have a notably lower percentage of pack rust occurrence, LaPorte and Greenfield. It is believed that this observation is due to bridge maintenance practices that include regular annual pressure washing of the superstructure.

The members which were observed to be affected by pack rust are listed below:

1. end diaphragms
2. gusset plates and connections
3. beam cover plates
4. cross bracings
5. hinge-pin connections
6. splice plates
7. bearings (rocker bearings and elastomeric bearings)

Pack rust occurrence in hinge-pin connections and gusset plate connections were observed to be present in more than 90% of the bridges with such details. Moreover, pack rust occurrence was less than 10% in end diaphragms, beam cover plates and cross bracing. The pack rust occurrence in rocker bearings was found to be around 30%, and for splice plate connections it was 13%. The pack rust occurrence in bearings (30%) and splice connection (13%) is less compared to gusset plate and hinge-pin connections (greater than 90%), but the number of bridges that have pack rust in bearings (318 bridges) and splice connections (214 bridges) are more than the number of bridges with pack rust in gusset plates (35 bridges) and hinge-pin connections (54 bridges). The observed occurrence of pack rust in bearings in the LaPorte District is half of that in the Fort Wayne District. It is believed that the reason for this discrepancy is because the LaPorte District pressure washes bearings and the superstructure with a waterjet to remove salt and debris every year.

The observed occurrence of pack rust in connection splices of bridges that intersect water bodies is higher than connection splices of bridges that intersect roads by 7% and railroads by 11%. When pack rust does occur, the trend line indicates that it takes 12 years—on average—after painting (mostly repainting) to initiate pack rust in splices and 32 years from the last paint contract to develop a severe pack rust condition. No correlation was found between pack rust occurrence in splices and vertical under-clearance.

The edge distance and the initial pretension in the bolts play a major role in preventing pack rust in splice connections and other connections. Large edge distances and low fastener pretension allow crevice edges to open and water to penetrate.

Experimental studies showed that stripe coated connections with the bottom crevice un-caulked experienced the least amount of corrosion and minimum pit depth for new structures. A second series of specimens involved plates that were corroded, cleaned, assembled and then stripe coated and caulked; the caulk that was placed on all sides was found to produce the best results (Shoyer et al., 2018).

5.2 Conclusions and Recommendations

Based upon the observations in this study the following recommendations and conclusions can be made:

The use of small edge distances with properly tightened high-strength bolts will keep material in firm contact and minimize crevice openings. The use of bolt stagger in new splice connections should be avoided.

Retain current INDOT provisions for stripe coating of new structures. Further study should be done to investigate the effectiveness of stripe coating and need to modify the number of stripe coats utilized.

Pack rust formation can be minimized in splice plate details, where no pack rust has been detected, if the connection region is cleaned and a stripe coat is applied along the crevice at a frequency of no more than 12 years. The opening between the flanges can be sealed with a suitable filler material to prevent entry of moisture. If rust bleeding is observed in splice connections, use of an alkaline penetrating sealer appears to be the best option.

If caulk is used to seal crevices, rust, debris and salts should be removed and the surfaces cleaned before caulking the crevice, or it should not be caulked. Caulking an active crevice corrosion cell will likely accelerate the corrosion process.

The use of penetrating sealers that are alkaline and has the appropriate viscosity to penetrate into crevices show promising results in mitigating pack rust. The crevice should be cleaned by mechanical tools or high-pressure water jets before applying penetrating sealers. Further study of these sealers should be considered to establish whether or not they should be used regularly in Indiana.

The washing of the bearings with pressurized water jets appears to be an effective maintenance practice which reduces the chances of pack rust occurrence in bearings.

REFERENCES

Ballinger, C., & Senick, W. (2003, September). Blasting is no way out. *Roads & Bridges, 41*(9), 48–49, 62.

Brown, B. F., Fujii, C. T., & Dahlberg, E. P. (1969). Methods for studying solution chemistry with stree corrosion cracks. *Journal of Electrochemical Society*, *116*(2), 218–219. https://doi.org/10.1149/1.2411799

Chiba, A., Nagataki, A., & Nishimura, T. (2016). Electrochemical aspects of interstitial nitrogen in carbon steel: Passivation in neutral environments. *Journal of the Electrochemical Society*, *164*(2), C17–C26. https://doi.org/10.1149/2.0541702jes

DeForce, B. (2010). *Revisiting the crevice corrosion of stainless steel and aliminum in chloride solutions—The role of electrode potential* (Doctoral dissertation). State College, PA: Penn State University.

Evans, U. R. (1948). *Metallic corrosion passivity and protection*. London, UK: Edward Arnold & Co.

FHWA. (2015). *Steel bridge design handbook, corrosion protection of steel bridges* (Publication No. FHWA-HIF-16-002-Vol.19). Washington, DC: U.S. Department of Transportation, Federal Highway Administration.

Fontana, M. G. (1987). *Corrosion engineering*. New York, NY: McGraw-Hill Education.

INDOT. (2018, April). *Winter operations districts and subdistrict details* [Memo]. Indianapolis, IN: Indiana Department of Transportation.

Jones, D. A. (1996). *Principles and prevention of corrosion*. Upper Saddle River, NJ: Prentice Hall.

Koch, G. H., Brongers, M. P., Thompson, N. G., Virmani, Y. P., & Payer, J. (2002). *Corrosion costs and preventive strategies in the United States* (Publication No. FHWA-RD-01-156). Houston, TX: NACE International.

Lansing Community College. [LCC Stars]. (2009, December 2). *Lansing Community College—Restoration and preservation of historic metals: Pack rust removal*. Retrieved from Youtube: https://www.youtube.com/watch?v=k3qC-pXYqqw

McCafferty, E. (1974). *Corrosion of iron in crevice*. Arlington, VA: Office of Naval Research.

NTSB. (1984). *Collapse of a suspended span of Interstate Route 95 Highway bridge over the Mianus River, Greenwich, Connecticut, June 28, 1983* (Highway Accident Report HAR-84-03). Washington, DC: National Transportation Safety Board.

Nystrom, E. A., Lee, J. B., Sagues, A. A., & Pickering, H. W. (1994). An approach for estimating anodic current distribution in crevice corrosion from potential measurements. *Journal of the Electrochemical Society*, *141*(2), 358–361. https://doi.org/10.1149/1.2054731

ODOT. (2015). *Oregon standard specifications for construction*. Salem, OR: Oregon Department of Transportation.

Pickering, H. (1993). Review of the IR induced mechanism of crevice corrosion. In *Corrosion control for low-cost reliability: 12th International Corrosion Congress* (pp. 1929–1937). Houston, TX: NACE International.

Pickering, H. W., & Frankenthal, R. P. (1972). On the mechanism of localized corrosion of iron and stainless steel. *Journal of Electrochemical Society*, *119*(10), 1297–1310. https://doi.org/10.1149/1.2403983

Shoyer, E., Ault, P., McDonagh, P., & Prazenka, B. (2018, January). Preventing crevice corrosion in new and existing steel structures. *Journal of Protective Coatings & Linings,* 20–32.

U.S. Climate Data. (n.d.). U.S. climate data. Retrieved May 2018 from https://www.usclimatedata.com/climate

APPENDIX

TABLE A.1
Detailed list of states with a mitigation strategy and specific details regarding them

States	Caulking	Size	Penetrating sealer	Stripe coat	Stripe coat sequence	New or existing coating	Comments
AL				×	spc - pc	new	
CA	×	>0.006″		×		existing	Different painting sequence for spot blast cleaning and for paint completely removed. Caulk sequence ambiguous
DE	×	≤1/2″	×	×	pc - spc	(new)	100% solid rust penetrating sealer for ≤1/2″
FL	×	>0.003″		×	pc - spc - ic - sic	new, existing	Caulking sequence not clear (probably after intermediate stripe coat)
GA				×	spc	new, existing	Sequence not mentioned
IL			×	×	pfc - fc	new	penetrating sealers used for spot painting
IN	×			×		new	Sequence not mentioned
IA	×	>3/16″	×	×	spc-pc	new, existing	cracks and seems <3/16″ if not effectively sealed by prime coat, caulking is required
LA	×	<1/2″	×	×	pc - spc	new, existing	
MD	×	>1/8″		×	sic - ic - sfc - fc	new, existing	
MA				×	spc - pc (new); sic - ic (existing)	new, existing	
MN				×	spc - pc	—	A national survey report prepared by KTA-Tator, Inc for MnDOT recommends Epoxy penetrating sealer/ epoxy mastic/ polyurethane finish for pack rust regions.
MO	×		×			existing	Calcium sulfonate rust penetrating sealer
NJ				×	spc - pc	existing	
NY			×	×	ic - sic	—	epoxy penetrating sealer
NC				×	ic - sic	—	
OH	×	>1/8″		×	pc - spc	new	
OR	×	>1/4″		×	spc - pc - sic - ic - fc	new, existing	pack rust removal practice also mentioned, caulking over baking material
PA				×	sic - ic - sfc - fc (new); spc - pc - sic - ic - fc (existing)	new, existing	remove pack rust by hand or power tool before abrasive blast cleaning
SD				×	spc - pc - sic - ic - sfc - fc	new, existing	
TN	×					new	Before painting, use silicone caulk to seal the top of splices of webs in girders without cover plates.
TX			×	×	pc - sic - ic - fc	new, existing	refer to word file for details
VA				×	pc - sic - ic - sfc - fc	new, existing	
WA	×	>1/16″ and <1/4″	×	×	pc - spc - ic - sic	new, existing	
WV	×			×	pc - spc	new, existing	Caulk applied after intermediate coat applied
WI				×	spc - pc or pc - spc	new, existing	

Note: Table A.1 lists which states specify caulking and their respective size limits as to when caulking can be used. It lists the states which specify stripe coating and the order of application of stripe coat with the painting system used in state. It also specifies whether the mitigation strategy is for new bridges or for existing bridges.

Notation: pc, primer coat; spc, stripe coat of primer coat; ic, intermediate coat; sic, stripe coat of intermediate coat; fc, finish coat; sfc, stripe coat of finish coat.

TABLE A.2
Breakdown of the number of bridges that intersect particular feature

District		Water	Road	RR	Abandoned RR
Fort Wayne	With pack rust	34	65	25	3
	Total	56	121	36	5
	Percentage	61	54	69	60
LaPorte	With pack rust	24	51	7	2
	Total	65	176	77	8
	Percentage	37	29	9	25
Crawfordsville	With pack rust	53	51	16	9
	Total	110	131	49	12
	Percentage	48	39	33	75
Greenfield	With pack rust	31	69	15	5
	Total	121	338	58	8
	Percentage	26	20	26	63
Seymour	With pack rust	45	34	10	3
	Total	129	123	27	7
	Percentage	35	28	37	43
Vincennes	With pack rust	45	30	15	0
	Total	129	99	52	4
	Percentage	35	30	29	0
Indiana	With pack rust	232	300	88	20
	Total	610	988	299	44
	Percentage	38	30	29	45

TABLE A.3
Number of bridges with pack rust in splice connections over given intersecting feature

District		Water	Road	RR	Abandoned RR
Fort Wayne	With pack rust	11	19	4	0
	Total	53	121	36	5
	Percentage	21	16	11	0
LaPorte	With pack rust	15	19	4	2
	Total	65	176	77	8
	Percentage	23	11	5	25
Crawfordsville	With pack rust	19	20	3	0
	Total	110	131	49	12
	Percentage	17	15	6	0
Greenfield	With pack rust	14	18	3	2
	Total	121	338	58	8
	Percentage	12	5	5	25
Seymour	With pack rust	19	10	2	1
	Total	129	123	27	7
	Percentage	15	8	7	14
Vincennes	With pack rust	29	18	6	0
	Total	129	99	52	4
	Percentage	22	18	12	0
Indiana	With pack rust	107	104	22	5
	Total	607	988	299	44
	Percentage	18	11	7	11

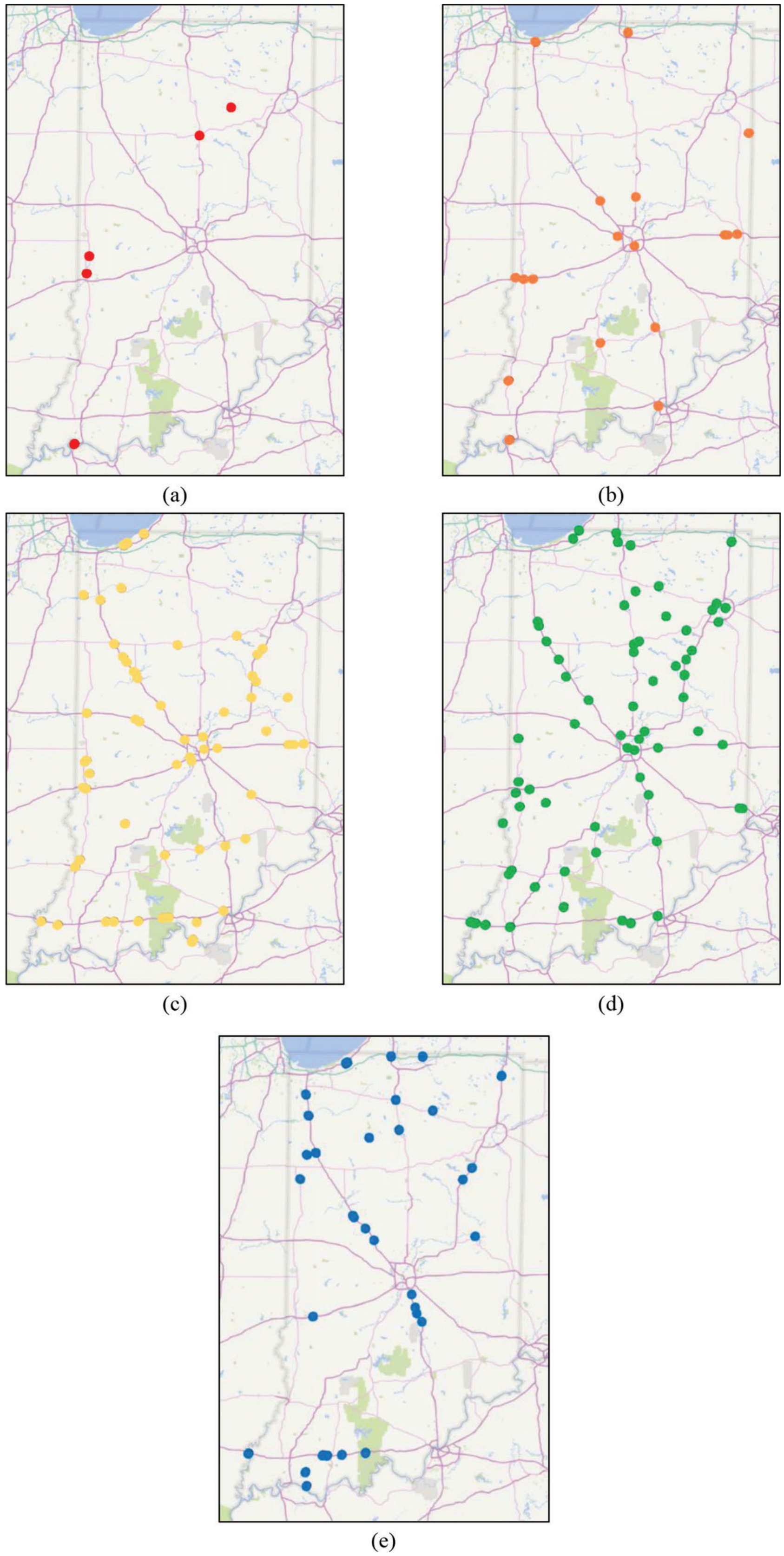


Figure A.1 Location of state owned steel bridges that have pack rust in splices (a) Rating 1, (b) Rating 2, (c) Rating 3, (d) Rating 4, (e) Rating 5.

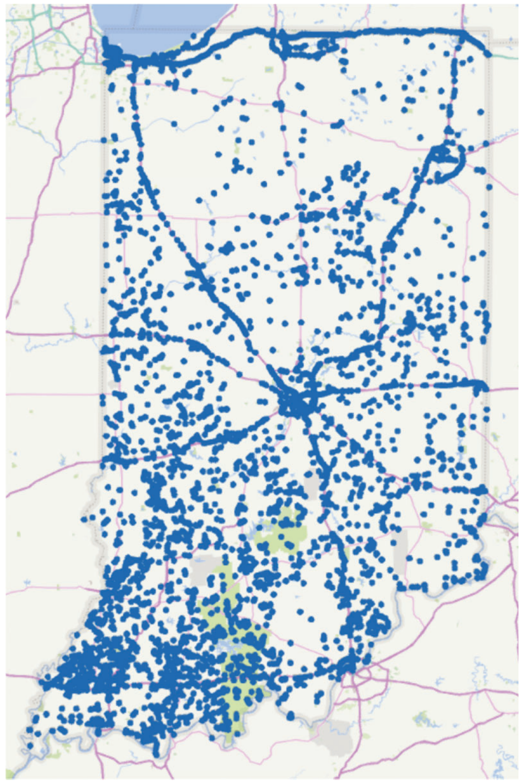


Figure A.2 Location of all steel bridges in Indiana.

About the Joint Transportation Research Program (JTRP)

On March 11, 1937, the Indiana Legislature passed an act which authorized the Indiana State Highway Commission to cooperate with and assist Purdue University in developing the best methods of improving and maintaining the highways of the state and the respective counties thereof. That collaborative effort was called the Joint Highway Research Project (JHRP). In 1997 the collaborative venture was renamed as the Joint Transportation Research Program (JTRP) to reflect the state and national efforts to integrate the management and operation of various transportation modes.

The first studies of JHRP were concerned with Test Road No. 1—evaluation of the weathering characteristics of stabilized materials. After World War II, the JHRP program grew substantially and was regularly producing technical reports. Over 1,600 technical reports are now available, published as part of the JHRP and subsequently JTRP collaborative venture between Purdue University and what is now the Indiana Department of Transportation.

Free online access to all reports is provided through a unique collaboration between JTRP and Purdue Libraries. These are available at: http://docs.lib.purdue.edu/jtrp

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