PACK RUST IDENTIFICATION AND MITIGATION STRATEGIES FOR STEEL BRIDGES

A Thesis

Submitted to the Faculty

of

Purdue University

by

Chintan Hitesh Patel

In Partial Fulfillment of the

Requirements for the Degree

of

Master of Science in Civil Engineering

December 2018

Purdue University

West Lafayette, Indiana

THE PURDUE UNIVERSITY GRADUATE SCHOOL
STATEMENT OF THESIS APPROVAL

Dr. Mark D. Bowman, Chair

Lyles School of Civil Engineering

Dr. Arun Prakash

Lyles School of Civil Engineering

Dr. Ayhan Irfanoglu

Lyles School of Civil Engineering

Approved by:

Dr. Govindaraju Rao

Lyles School of Civil Engineering

To my Family.

ACKNOWLEDGMENTS

I would like to thank to my Professor and Advisor Dr. Mark Bowman for giving me an opportunity to work on this project. I am very grateful for his constant motivation and guidance throughout my graduate program.

I am grateful to the following Research Study Advisory Committee members from INDOT for their support during this study: Tim Wells, Stephanie J. Wagner, Christopher Wheeler, Jose Ortiz, Michael Black, and Nathaniel Pfeiffer.

I am thankful to Dr. Arun Prakash and Rafael Armendariz Briones for helping and guiding me in the finite element analyses study. I would also like to thank my committee members Dr. Arun Prakash and Dr. Ayhan Irfanoglu for providing their valuable time in reviewing my work and providing valuable advice to enrich my work.

I would also like to thank my family and friends for their constant support.

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SYMBOLS

E_{surf}	Potential at the exterior open surface
E_x	Potential at a distance x from mouth of crevice
E_{pass}	Potential at which rate of dissolution changes drastically
I	Current flow from external surface into the crevice
R	Resistance provided by the electrolyte
$\Delta\phi^*$	Difference between potentials E_{surf} and E_{pass}
x_{active}	Distance from the mouth of the crevice to the point where active dissolution ends
L	Penetrating depth of a Penetrating sealer

ABBREVIATIONS

DOT	Department of Transportation
INDOT	Indiana Department of Transportation
BIAS	Bridge Inspection Application System

ABSTRACT

Patel, Chintan Hitesh M.S.C.E., Purdue University, December 2018. Pack Rust Identification and Mitigation Strategies for Steel Bridges. Major Professor: Mark D. Bowman.

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.

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 the bridges to have unobstructed traffic flow. Structural steel was one of the primary material 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 1950's to 1960's are at the end of their original intended service life. Many of these bridges require serious attention. 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 percent of the structurally deficient bridges are deficient because of corrosion [1]. These types of corrosion are visible on the surface. Crevice corrosion, however, is often not visible until it gets very severe, and this can lead to serious problems.

Pack rust has been a serious topic of research 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 1.5 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. [2] [3]

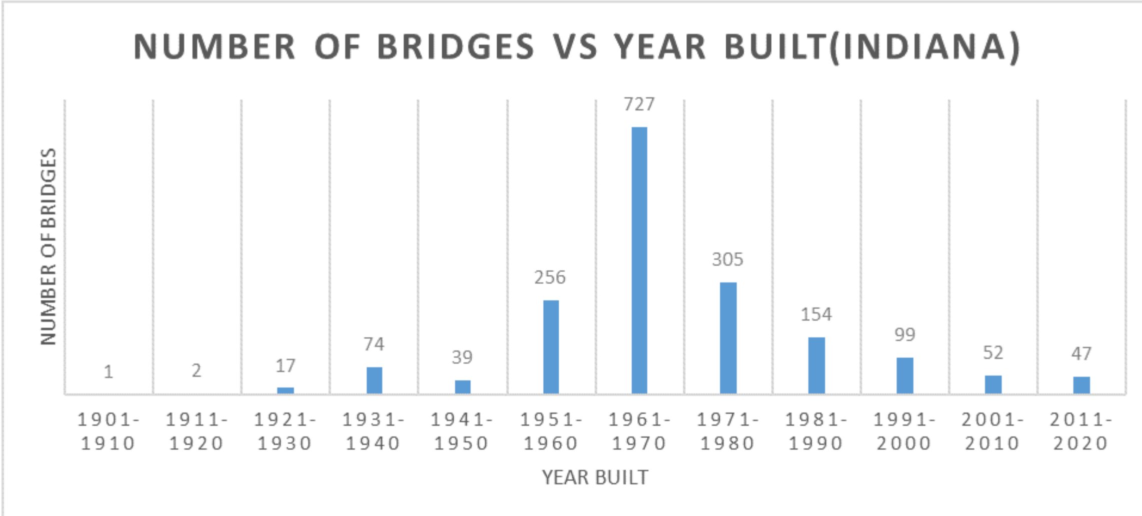

Fig. 1.1. Distribution of the steel bridges built in Indiana

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 used by other state DOTs
3. Documenting the occurrence of pack rust in Indiana bridges. This task involves answering the following question
   (a) Does pack rust occur frequently in bridges 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 BIAS system were used. It should be emphasized that all pack rust identification was done from the inspection reports and not by site inspections. A database of bridges with pack rust was created in 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.

2. LITERATURE REVIEW

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. [4]

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. [1]

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 percent of structurally deficient bridges. [1] 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

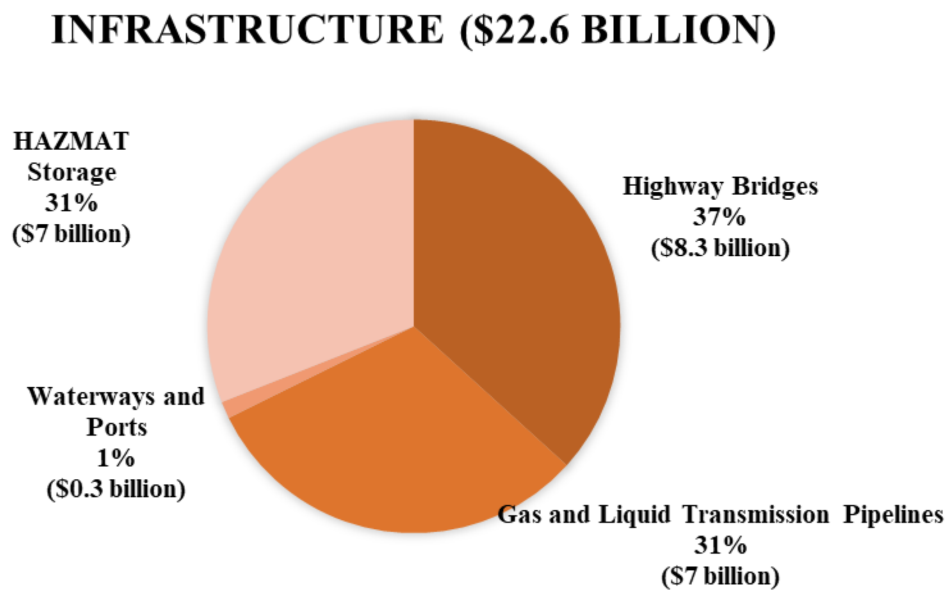

Fig. 2.1. Cost of corrosion in the infrastructure industry (Koch, Brongers, Thompson, Virmani, & Payer, 2002)

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 costs due to traffic delays, long detours that add to more fuel consumption, and wear and tear of vehicles. [1]

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 [4]. 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

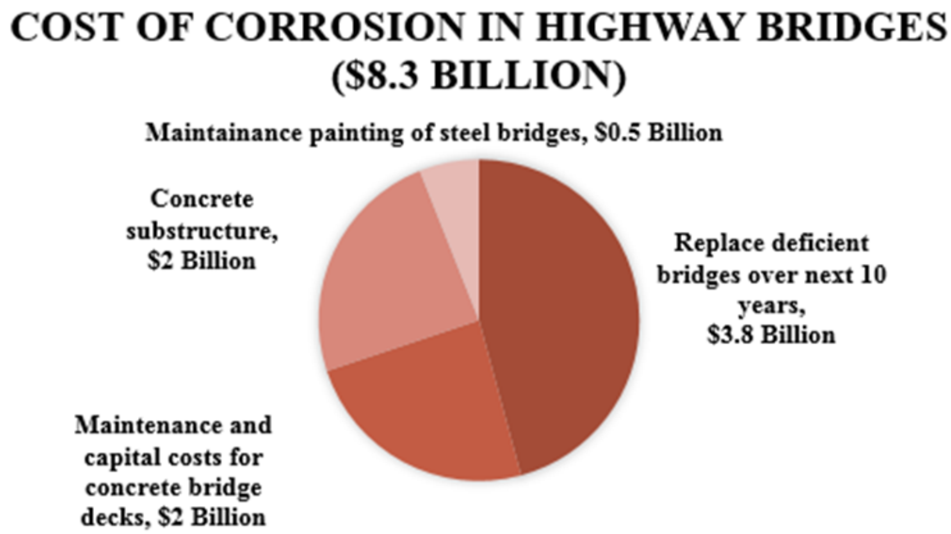

Fig. 2.2. Cost of corrosion in Highway Bridges(Koch, Brongers, Thompson, Virmani, & Payer, 2002)

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
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. Contact between elastomeric bearing pads and steel sole plate is an example where crevice corrosion can occur between a metal and non-metal. 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 today. 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). The second theory is based on the potential drop model. Neither theory is not able to prove all the observations seen in the actual crevice corrosion process.

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:

Anode: $\mathrm{M} \longrightarrow \mathrm{M}^{n+} + ne^-$

Cathode: $2\,\mathrm{H_2O} + 4\,\mathrm{e}^- + \mathrm{O_2} \longrightarrow 4\,\mathrm{OH}^-$

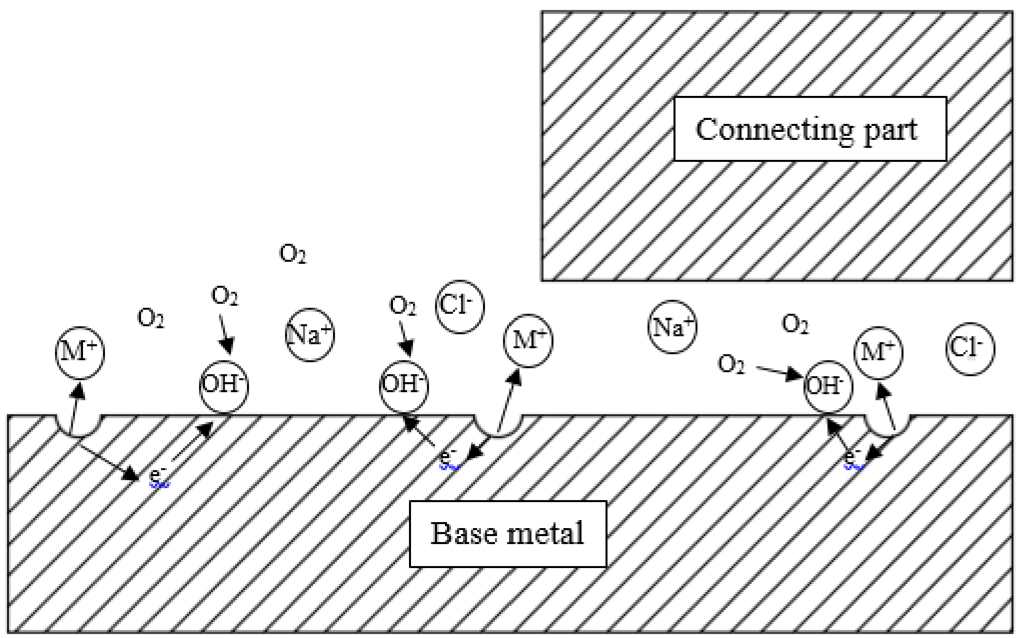

Fig. 2.3. Initial stage of corrosion

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 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 gets alkaline. The pH of the crevice solution gets stabilized at around pH 3 pH 4 since hydrolysis is thermodynamically unfavorable below this pH. [5]

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 gets 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. [6] Crevice corrosion process is unstable at pH 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 pH remains constant. [7]

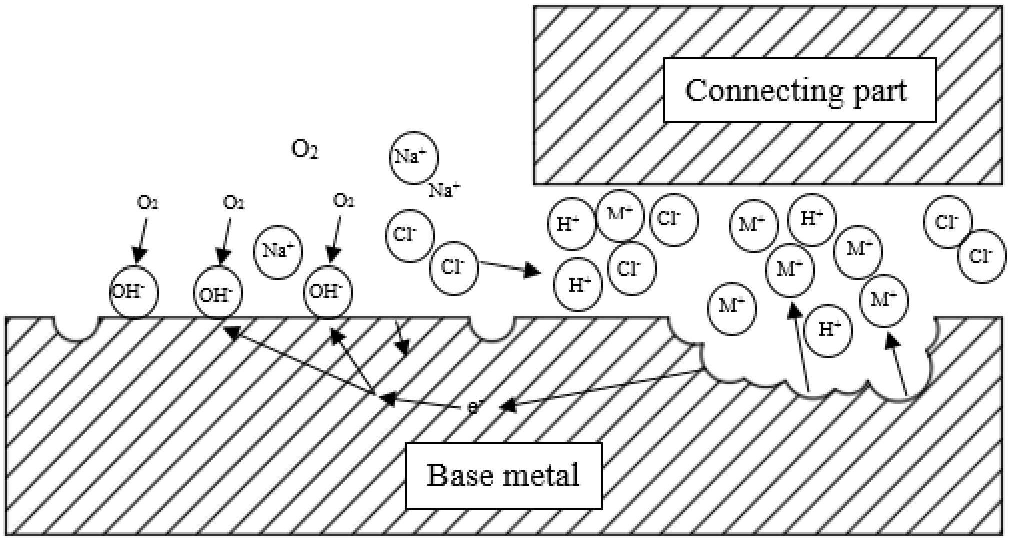

Fig. 2.4. Corrosion at a later stage

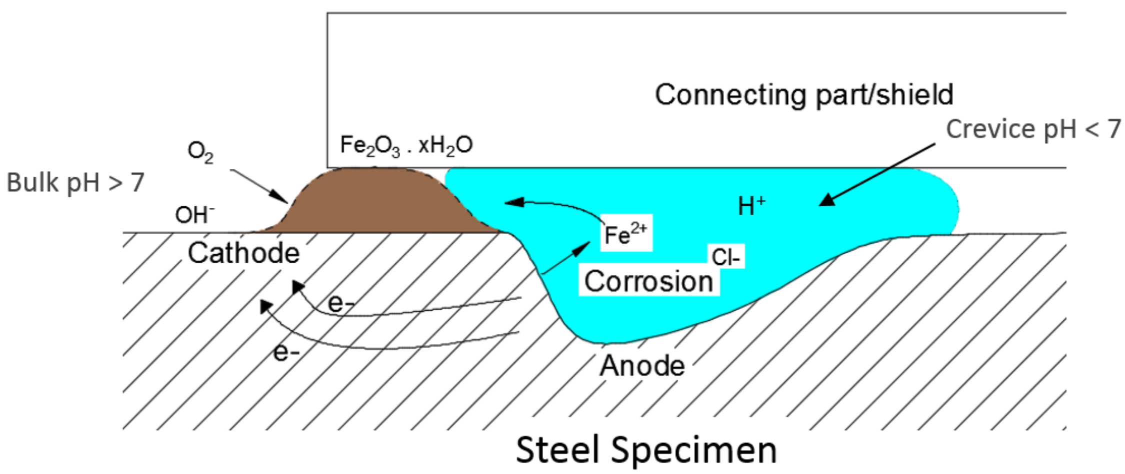

Fig. 2.5. Rust deposit at the mouth of the crevice and acidification of crevice solution

Theory 2

The second theory is based on the IR voltage and this theory was developed by Pickering and Frankenthal (1972) [5] when 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 [8].

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 [8].

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

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 $\Delta\phi^*$, see Figure 2.6. Therefore, $IR \geq \Delta\phi^*$ is the condition for the crevice corrosion to occur. [7] Figure 2.6 on the left shows the experimental setup at Pennsylvania State University for the crevice corrosion and on right crevice polarization curve. The shape of the corroded region closely resembles the polarization curve [9].

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 traditional mechanism was not able to do, 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

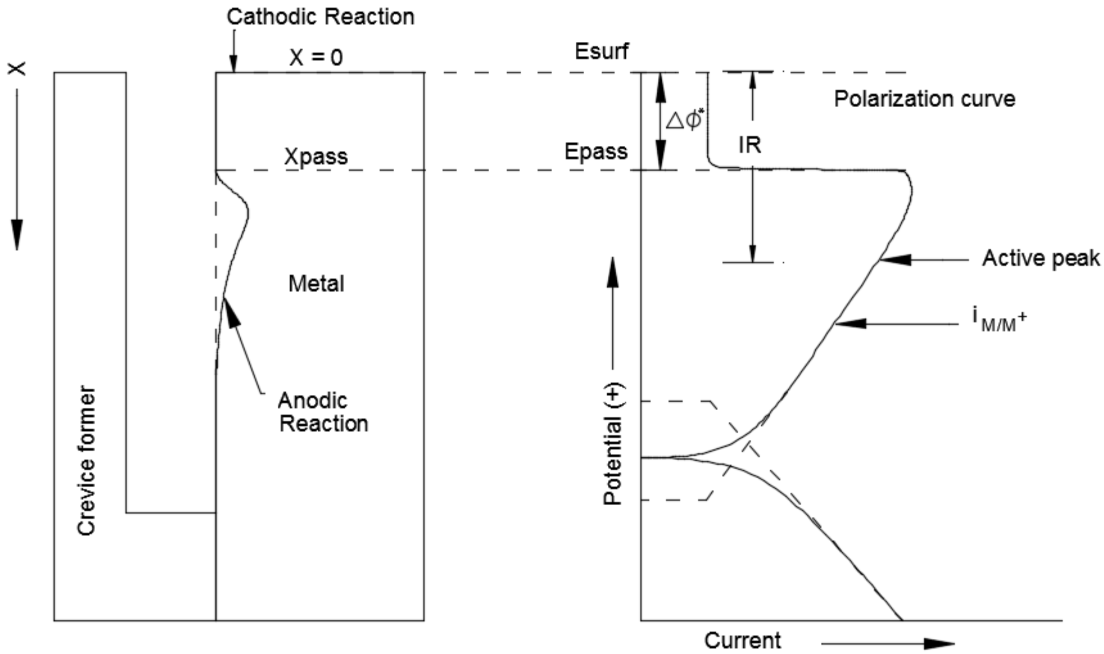

Fig. 2.6. Schematic of metal with crevice on the left and its matching polarization curve on the right [9]

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.

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.

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. [10]

The experiments conducted by the Naval Research Laboratory 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. [11] A plasma nitride treatment on the components of the bridge which will be in contact can prevent crevice corrosion. However, this method would be very uneconomical for application on bridges and will require detailed study in the field of crevice corrosion.

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 that 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 the 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 coat

2. Caulking

3. Penetrating Sealer

4. Backer rod

3.2.1 Stripe coat

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 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, 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

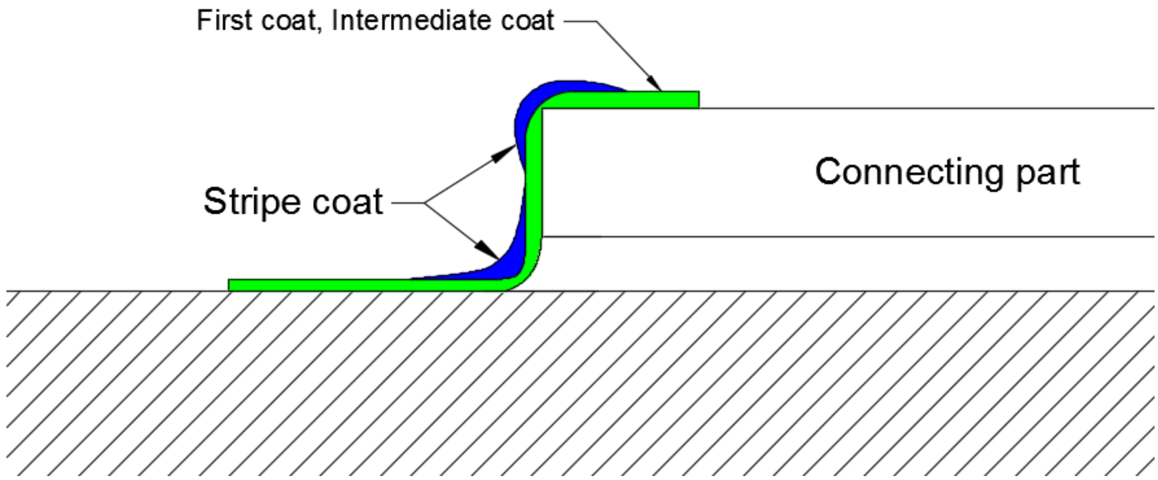

Fig. 3.1. Paint film and stripe coat thickness

expansion of adjoining components. Environmental factors and time will also impact the paint and cause cracking. The development of the 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, but 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 typical painting cycle. All this gives an impression that stripe coating cannot serve as a one-time, long-term mitigation strategy.

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 few years. However, caulking may be a bad choice where pack rust already exists and caulking is applied without any treatment. Ballinger and Senick [12] found that if an ongoing crevice corrosion cell is sealed the corrosion process accelerates by 400 fold.

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. 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. x_{active} is the distance from the mouth of the crevice to the point where active dissolution end, and L is the depth to which penetrating sealer can penetrate as shown in Figure 3.2. So for penetrating sealers to work effectively in preventing crevice corrosion; L should be greater than x_{active} ($L \geq x_{active}$).

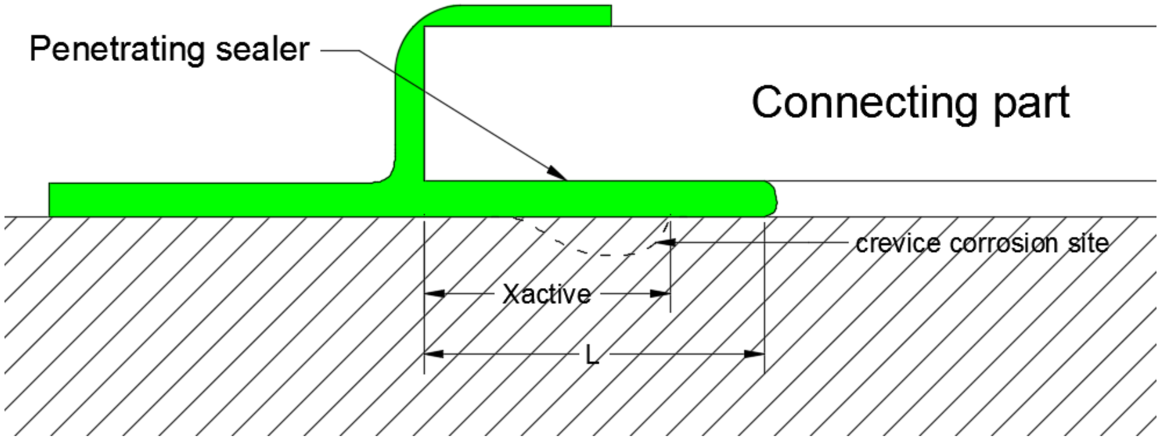

Fig. 3.2. Schematic diagram for penetrating sealers

There are eight states which recommend penetrating sealers in their painting specifications. Three states mention specific requirement 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 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.

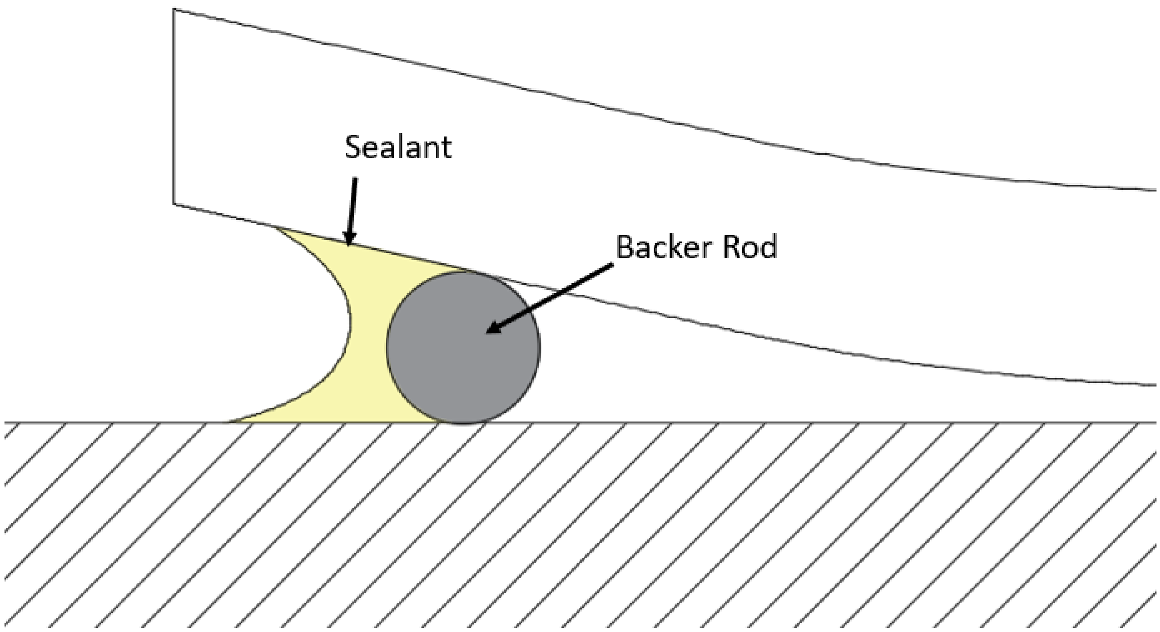

Fig. 3.3. Schematic diagram for backer rod and sealant

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

Table 3.1.
Summary of the states using mitigation strategies for pack rust

Stripe coating (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

3.3 Repair procedure by Oregon DOT

The Oregon Standard Specifications for Construction, 2015 outlines a method that should be used 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 plated with the rivet gun [13].

3.4 Industry Effort

Experiments were conducted by Shoyer et al. [14] to test the effectiveness of the 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 [15] and SSPC SP-11 [16] specifications. The new steel specimens were tested for specimens primed before assembly and primed after assembly of the members forming 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 put through accelerated corrosion 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 [14].

4. PACK RUST IN INDIANA BRIDGES

4.1 Procedure utilize for study

Consistent with federal requirements, each bridge in Indiana is inspected every two years, and the inspection reports are uploaded to the Bridge Inspection Application System (BIAS) 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 provide some evidence on whether or not pack rust is a problem in the state of Indiana. A data set 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:

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

For the bridges with pack rust in splices, latitude and longitude were also noted. The 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 phase of crevice corrosion, often could be detected if close-up images are 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 more number of photographs of the bridge than other districts, capturing every minute detail. 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 does not look good. 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.

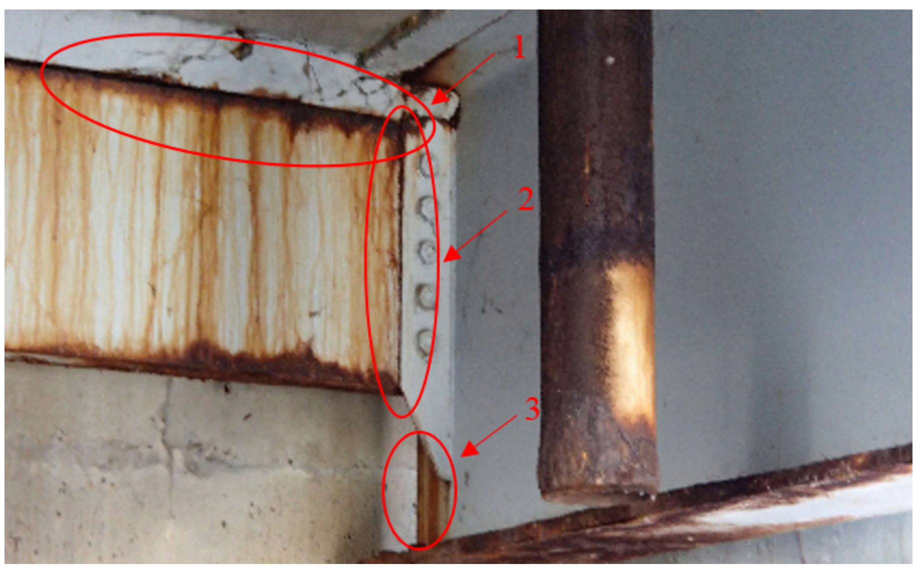

Fig. 4.1. Typical camera angle for photos of end diaphragms

Figure 4.2 shows pack rust in bridge number (I465)31-49-04449 B. Pack rust is 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 significantly large near the bolts. If the thickness of pack rust at the line of bolts is even 1/4-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 quality and number of images present in the inspection reports.

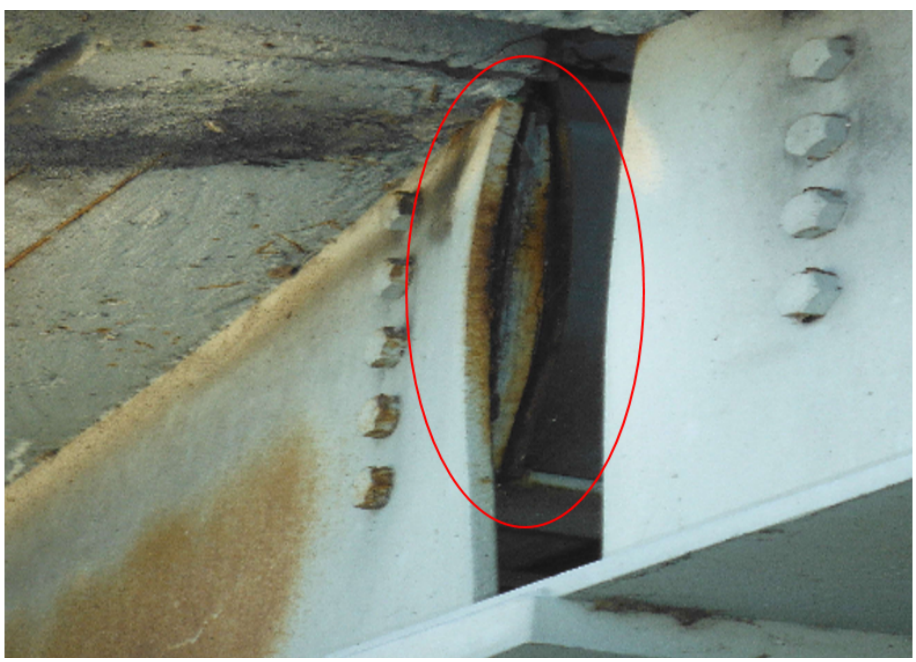

Fig. 4.2. Most severe pack rust in the diaphragm found in Indiana

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.

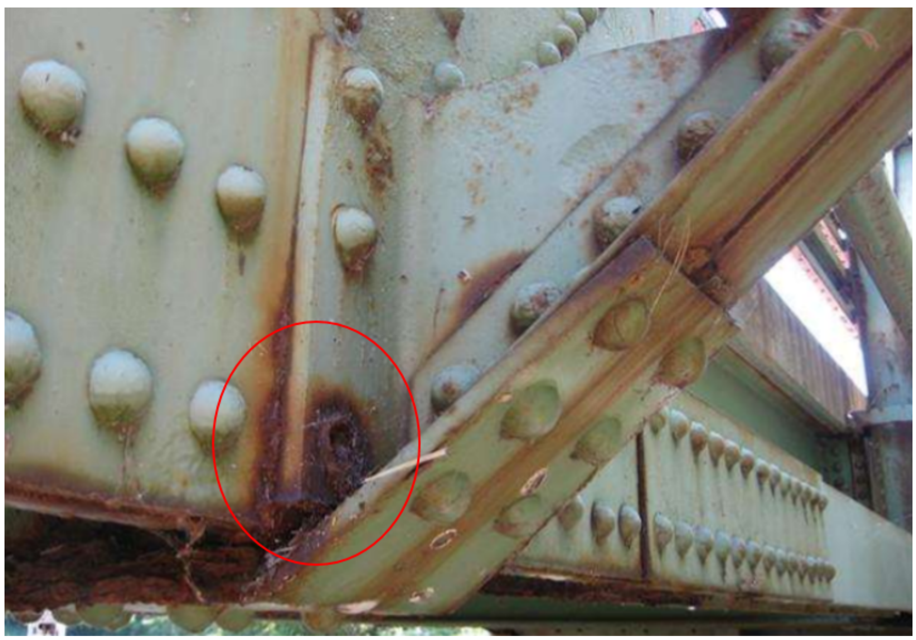

Fig. 4.3. Pack rust in the lower chord and gusset plate

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.

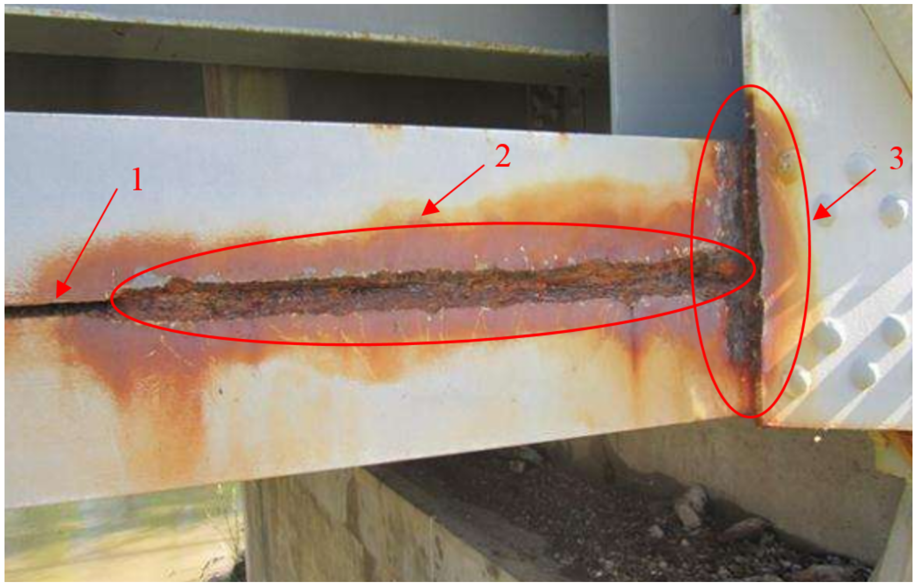

Fig. 4.4. Rivet failed due to pack rust

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 Figure 4.5. There are chances that this is not an active corrosion. No field testing was conducted for this study.

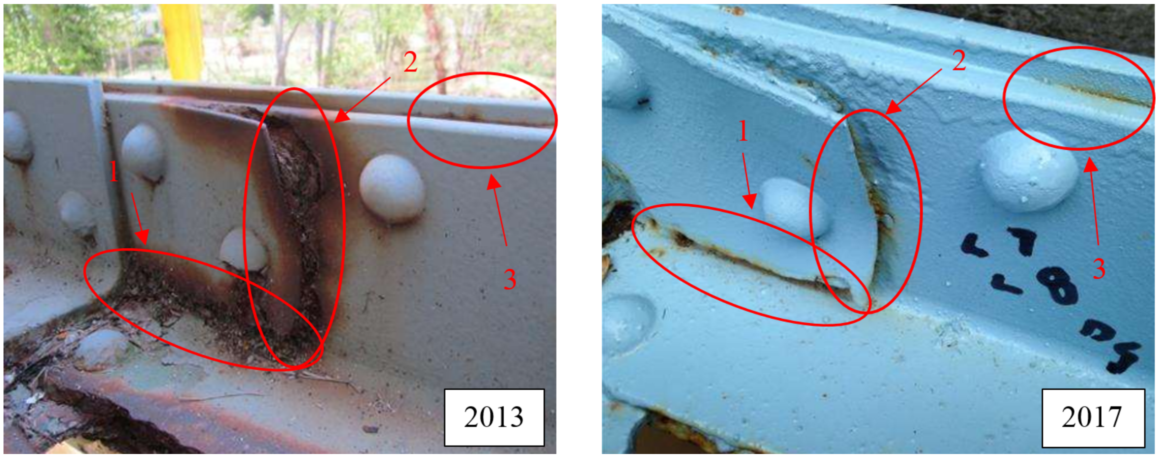

Fig. 4.5. Pack rust in connections of the bottom chord

4.2.3 Beam Cover Plate

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 inspectors markings for the weld cracks on the bridge to check crack propagation in 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.

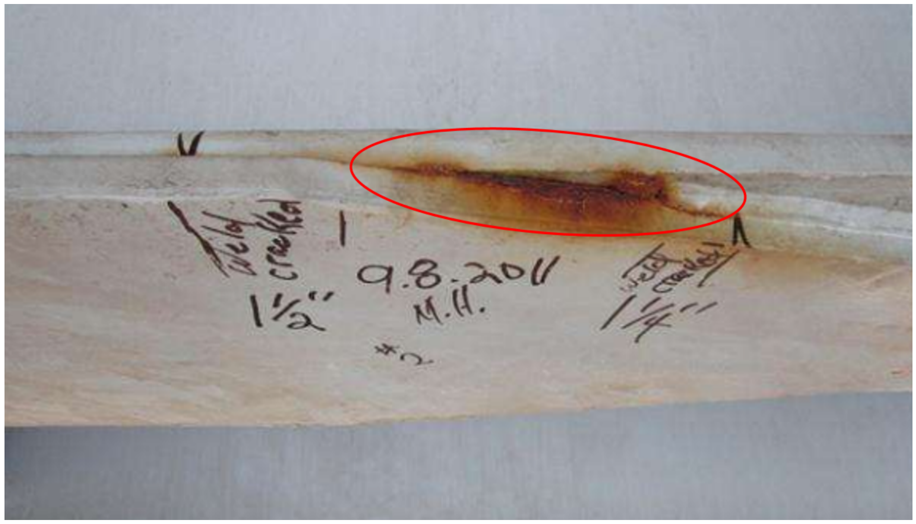

Fig. 4.6. Pack rust at the tapered location for wider cover plates (location 1)

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.

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

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.

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

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

4.2.4 Cross Bracing

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 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.

Fig. 4.10. Example of pack rust in cross bracing

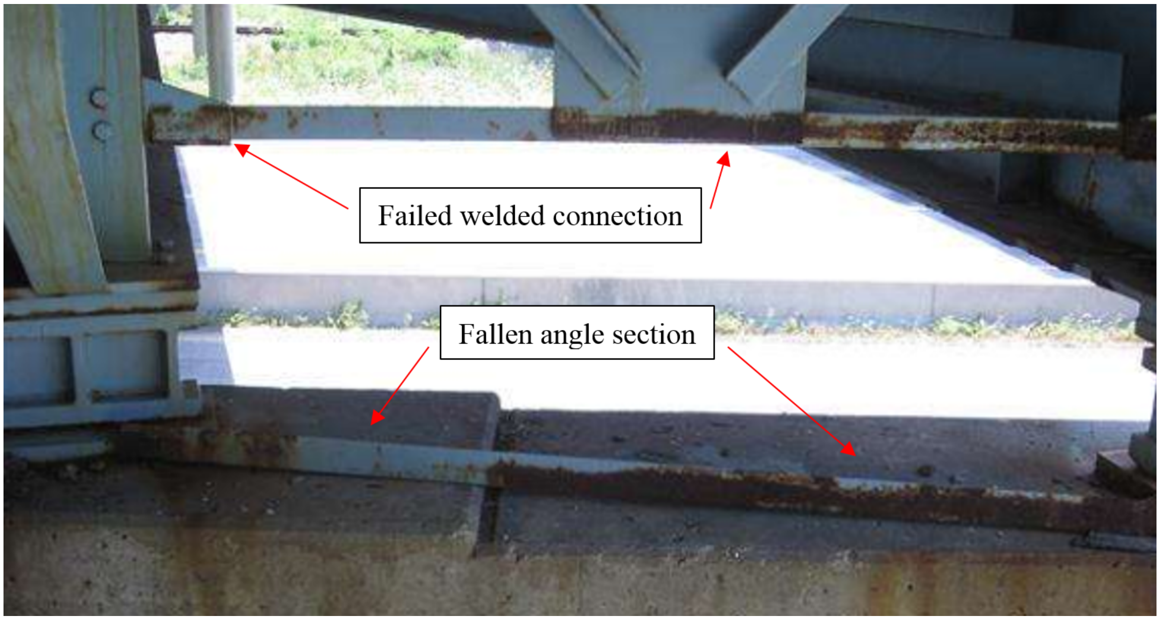

Fig. 4.11. Angle section fallen down because connection got failed due to pack rust

4.2.5 Hinge-pin connection

Hinge-pin connections are the 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 on the 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 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.

Fig. 4.12. Finger joint

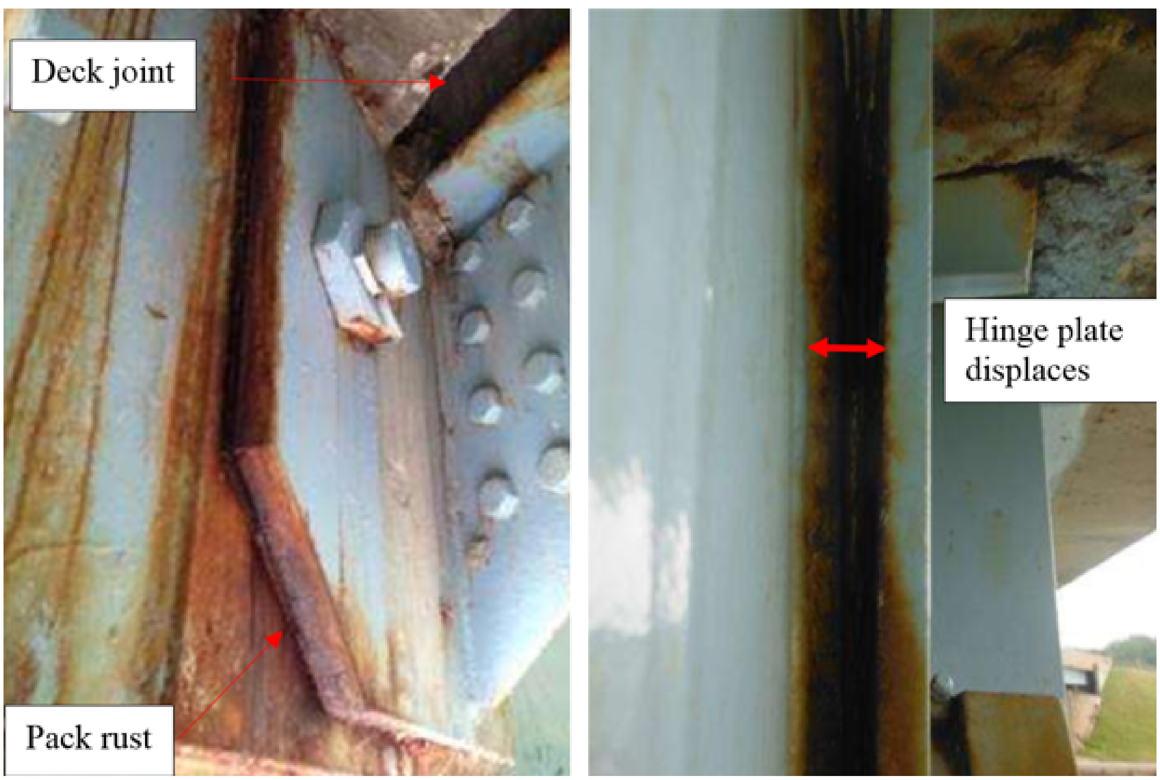

Fig. 4.13. Pack rust in the hinge-pin connection

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. 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 splice plate bends in excess causing large strains in bolts, they can 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

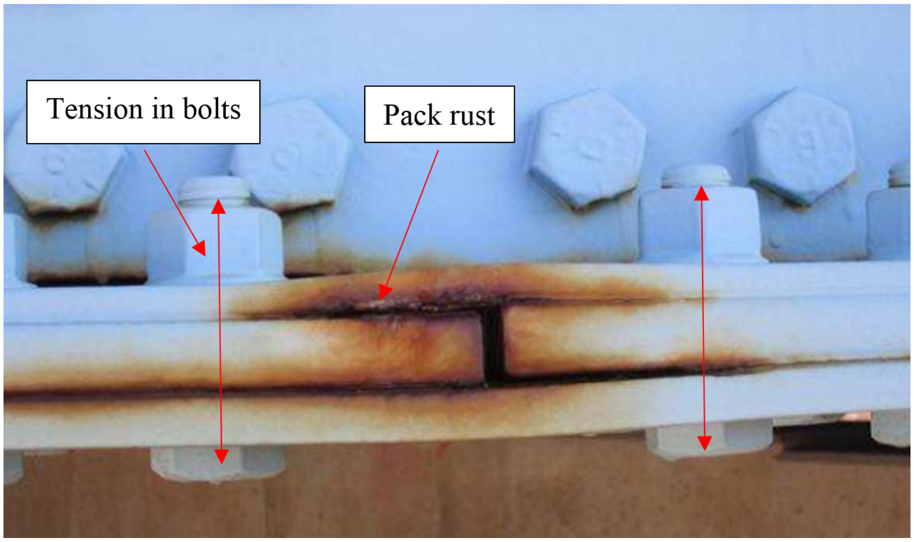

Fig. 4.14. Pack rust in the middle of the splice plate (location 1)

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

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

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

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 both the bearing types which made it difficult to identify and judge whether pack rust is present or not. Bearings have a very close relationship with the 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.

Fig. 4.17. Pack rust in the rocker bearings

Even though pack rust in rocker bearings occur quite frequently, mitigating pack rust in rocker bearings is not a high priority since INDOT plans to eventually re-

Fig. 4.18. Pack rust in elastomeric bearings

construct all the rocker bearings with semi-integral bearings, which are not prone to pack rust formation.

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 La Porte, middle includes Crawfordsville and Greenfield, and south includes Seymour and Vincennes.

For the districts Fort Wayne, Crawfordsville, and Greenfield, the number of bridges with pack rust is in close range; moreover, the number of bridges with pack rust are in close range for the districts La Porte, Seymour, and Vincennes (refer to Figure 4.19). It is also essential to compare the percentage of bridges in a district that has 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 percent of bridges in Greenfield have pack rust, which is least among all the districts in Indiana.

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 La Porte and Greenfield do not seem to follow the expected trend. The expectations were

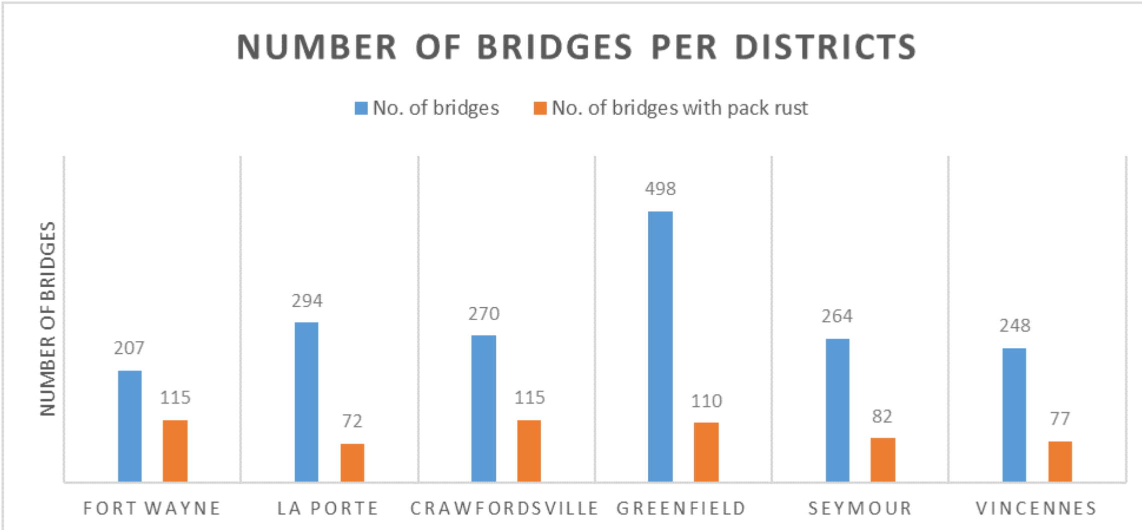

Fig. 4.19. Total number of bridges and number of bridges with pack rust in different districts

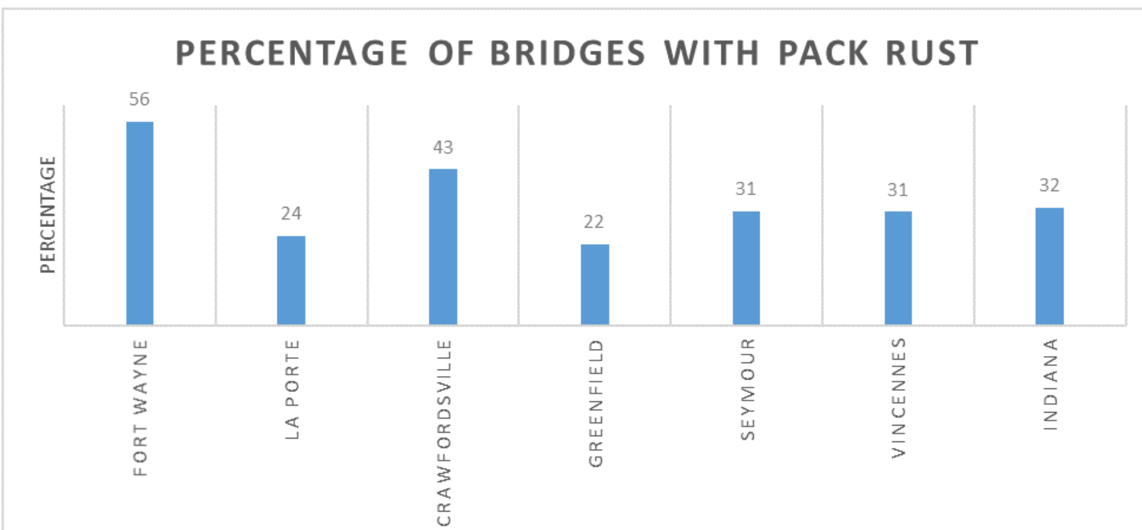

Fig. 4.20. Percentage of bridges with pack rust in districts and Indiana

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 [17], salt usage per lane miles and brine usage in columns 3&4. [18] This table provides information that was used to identify correlations between the salt usage and the pack rust percentage from north to south.

The average annual snowfall [17] in La Porte 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 in the two northern districts has a large difference due to the lake effect snowfall that occurs in the La Porte District. Although the snowfall in La Porte is twice of that in Fort Wayne, the salt usage in La Porte is only 15 percent more. The less salt usage than required is compensated by higher brine usage in the La Porte. However, the pack rust percentage in La Porte 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, La Porte 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 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.

The lower pack rust percentage observed in the La Porte and the Greenfield Districts may be the result of a good maintenance program in both the districts. Discussion with the Study Advisory Committee members confirmed that both of these district utilize 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.

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 years. avg.)	Brine Used (Gal.) (5 yrs. Avg.)	Pack rust percentage
Fort Wayne	34	11.1	119,673	56
La Porte	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

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 separated into four only 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 the 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.

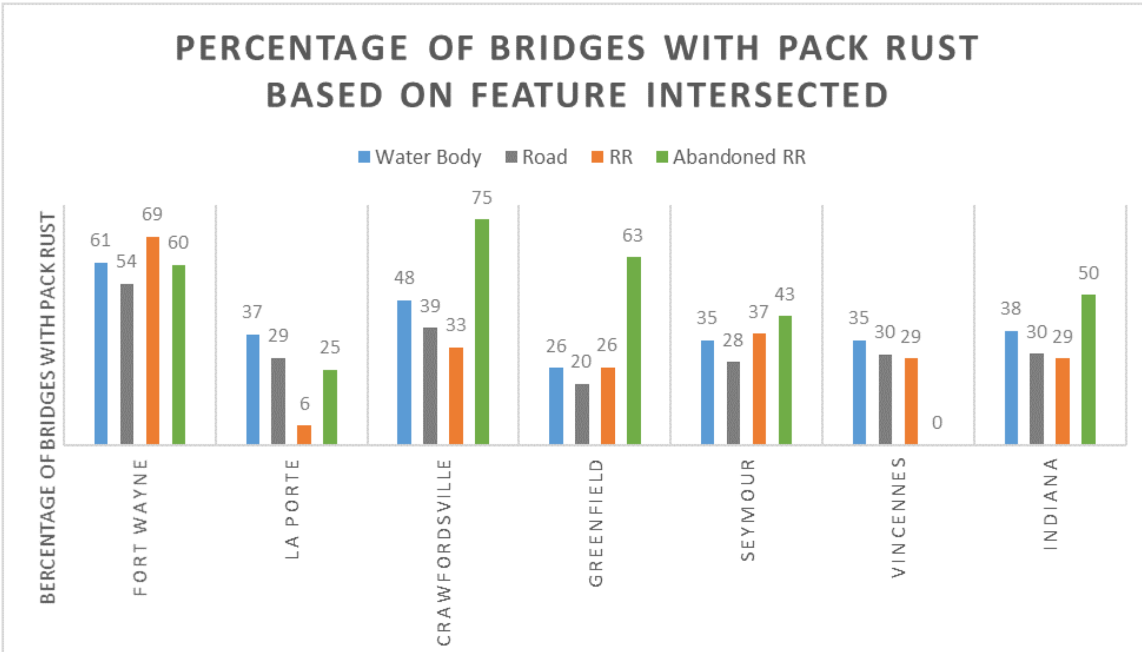

Fig. 4.21. Percentage of bridges with pack rust based on features intersected

In Figure 4.21, the percentage of the bridge with pack rust that intersects abandoned railroad 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 crossing abandoned railroads in Indiana. The number of bridges that have pack rust and the total number of bridges that intersect a 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 numbers are small. In general, the trends in the individual district 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 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., La Porte, 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 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, but the joint condition does not typically influence pack rust formation in splices.

Figure 4.22 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 La Porte 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 La Porte has a dedicated maintenance crew that annually pressure washes the bearings from all the dirt and salts using water jets.

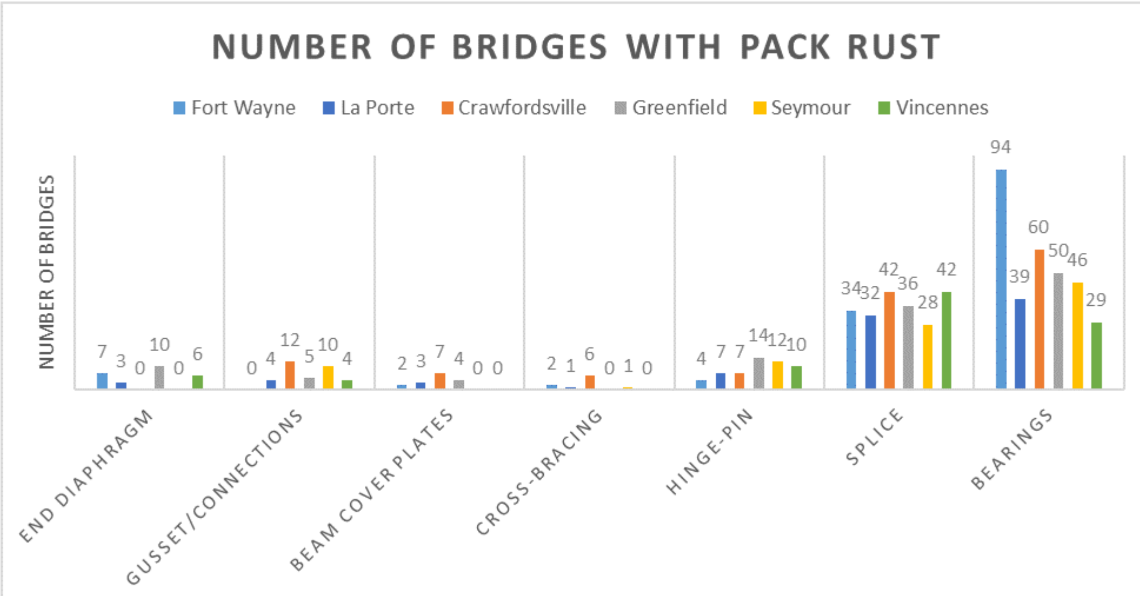

Fig. 4.22. Number of bridges with pack rust in bridge members

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 4.22, and so are the percentage of bridges that have pack rust. The number of bridges with rusted end diaphragms is large, but it is difficult to identify pack rust between the diaphragm and the connection plate, as well as the space between diaphragm elements (such as a pair of angles), so 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 the cover plate is 58. Therefore, the occurrence of pack rust in Crawfordsville district in cover plates is about 12 percent.

The gusset plates and other connections are generally used in truss bridges. A truss bridge has a lot of connections and gusset plates. All the truss bridges have pack rust in gusset plates, but not all the gusset plates in a specific bridge have pack rust. The most common place in truss bridges where pack rust occurs is the connections in the bottom chord. The washed away salt from the deck is the primary factor causing aggressive pack rust in the bottom chord of the 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 percent. The reason for this observation can directly be correlated with the fact that there are generally deck joints present directly above the hinge-pin connections, which allows water and salt to spill down onto this connection.

The total number of bridges with pack rust in splices and bearings stands out from the rest of the details. Figure 4.23 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 La Porte 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

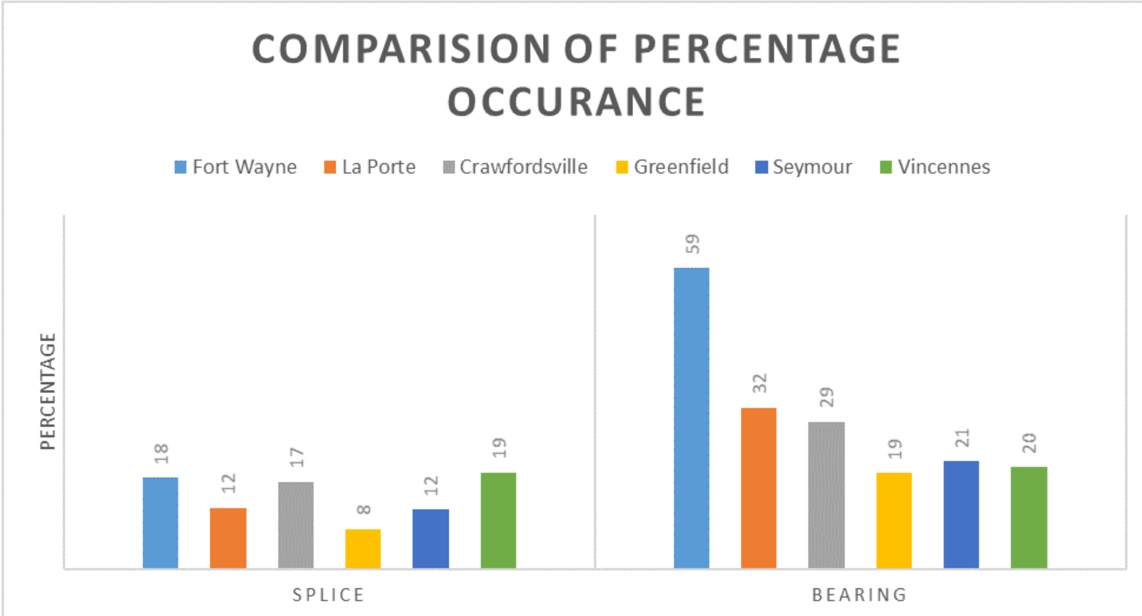

Fig. 4.23. Percentage of pack rust occurrence in splices and bearings for each district

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 pack rust at the bearings. It can be observed that the rating improve by a very small value as northern districts are compared with southern districts.

Table 4.2.
Average joint rating for the bridges with pack rust

District	Avg. Joint Rating for the bridges with pack rust in bearings
Fort Wayne	4.16
La Porte	4.2
Crawfordsville	4.22
Greenfield	4.22
Seymour	5.26
Vincennes	4.6

4.3.4 Pack Rust in Splice connection

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.

Pack rust ratings for splices

A rating system was developed to 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.

Table 4.3.: Pack rust rating for splices

Pack Rust (PR) Severity Rating		
Rating	Description	Examples
1	Severe PR: >3/4 inch bowing of splices or bolt failure	 (a)  (b)

Table 4.3 continued from previous page

		 (c)
2	Moderate to severe PR: 1/4 inch to 3/4 inch bowing of splice plates	 (d)
3	Moderate PR: visible bowing of the splice plates, <1/4 inch	 (e)

Table 4.3 continued from previous page

4	Minor to moderate PR: visible corrosion in middle, edges, or corners of the splice connection	 (f)
5	Minor PR: Rust bleeding in middle, edges, or corners of the splice connection	 (g)

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 4.24 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 which were not recorded, and unaccounted in this study.

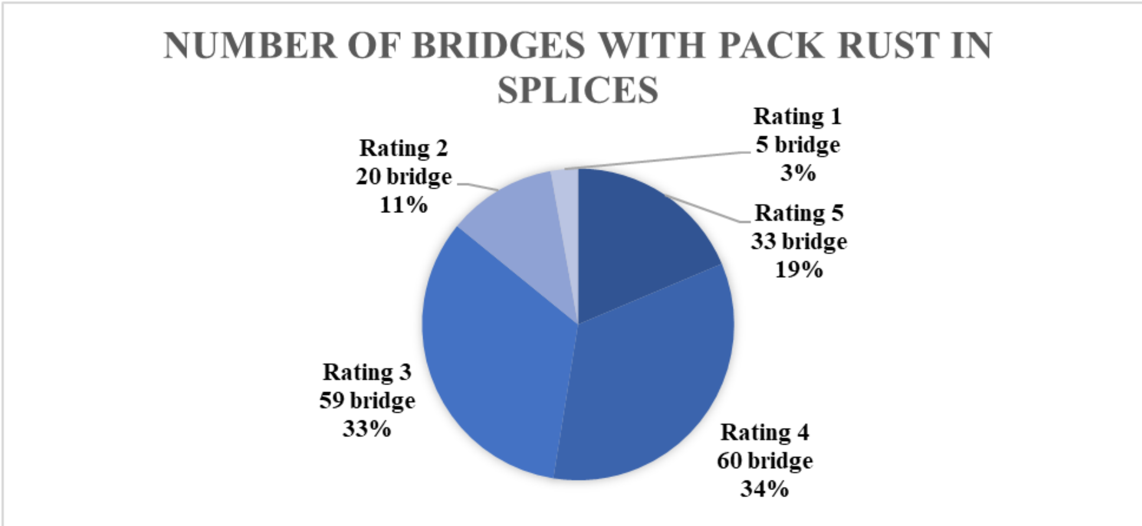

Fig. 4.24. Number of bridges with pack rust in splices classified by pack rust rating

Location of splices that have pack rust

The section illustrates which splices are most affected by pack rust. Figure 4.25 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.

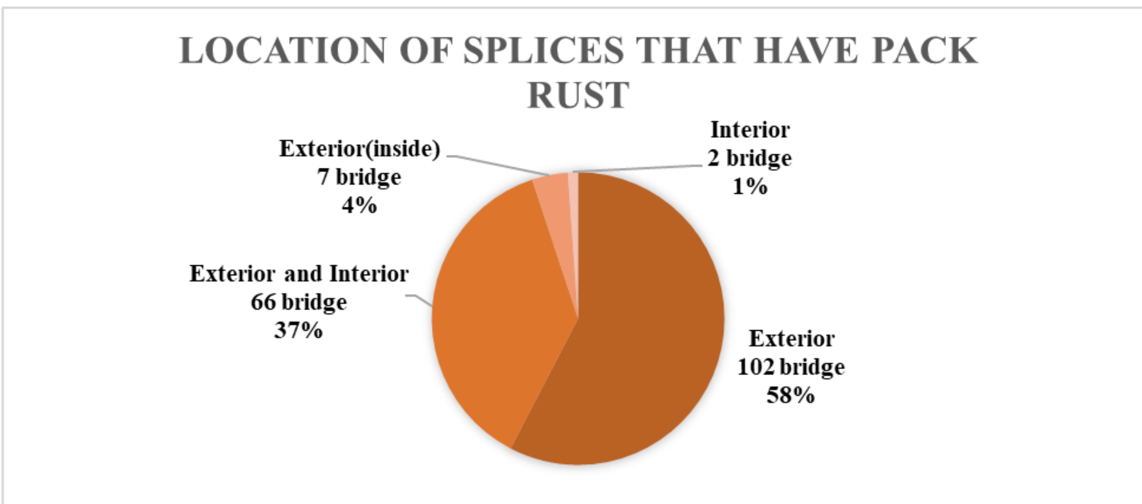

Fig. 4.25. Location of splice connections that have pack rust

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 4.26 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 given feature is tabulated in Appendix Table A.3.

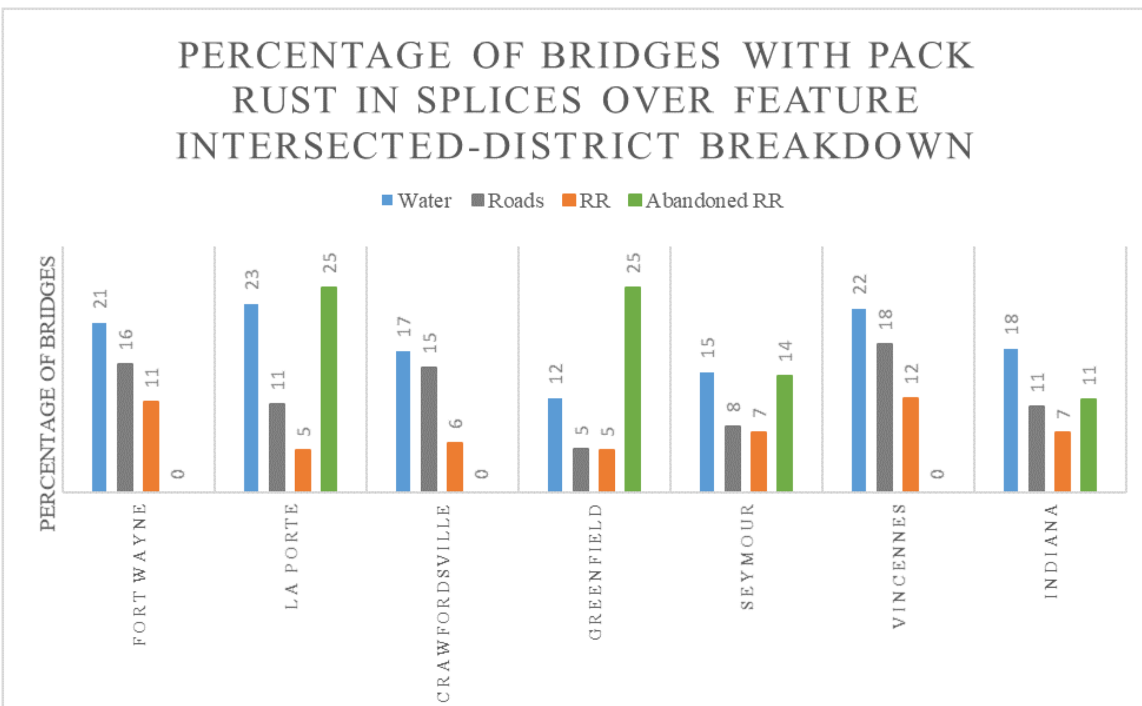

Fig. 4.26. Percentage of bridges with pack rust in splice over feature intersected

The average rating for the bridges which intersect a particular feature is shown in Figure 4.27. 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.

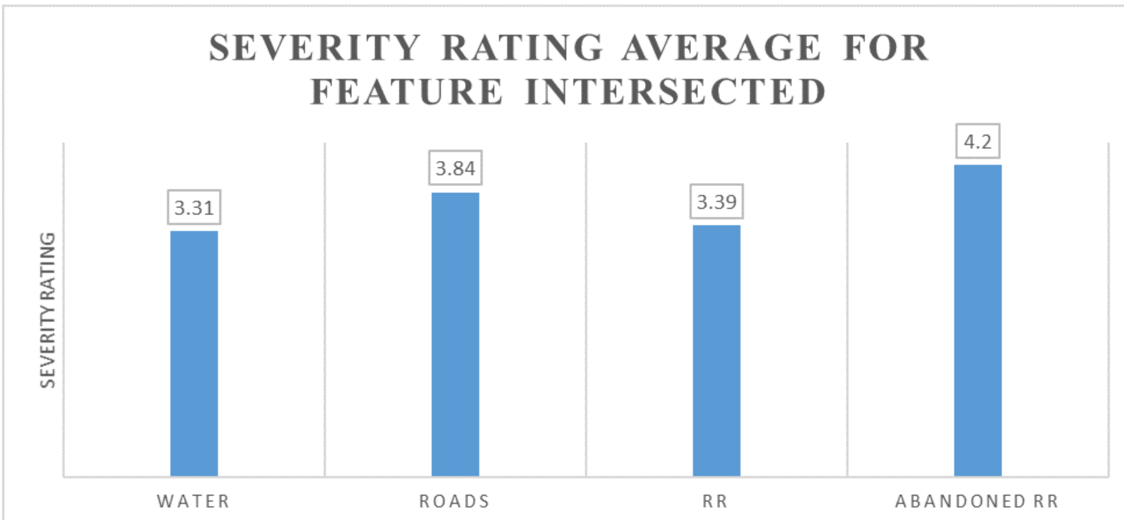

Fig. 4.27. Average severity rating for the intersecting feature

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 4.28. This is done to 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 for 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, there is 13 percent chances for the pack rust to occur in splice connection. There is a lot of scatter in the data (R2 = 0.29), with some bridges taking more than 25 years after painting to form minor pack rust in splices to just 10 years to reach very severe pack rust.

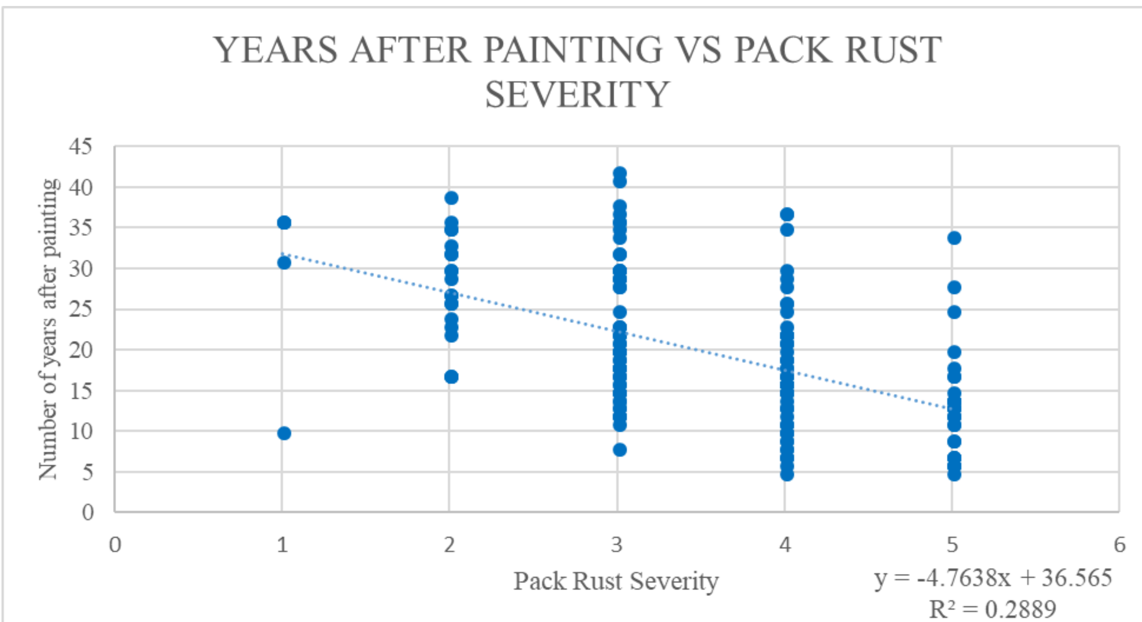

Fig. 4.28. Relation between years after painting and pack rust severity in splices

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 intersecting feature. The Figure 4.29 shows that pack rust of severity 3 in bridges intersecting roads reaches before it reaches in the bridges intersecting 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.

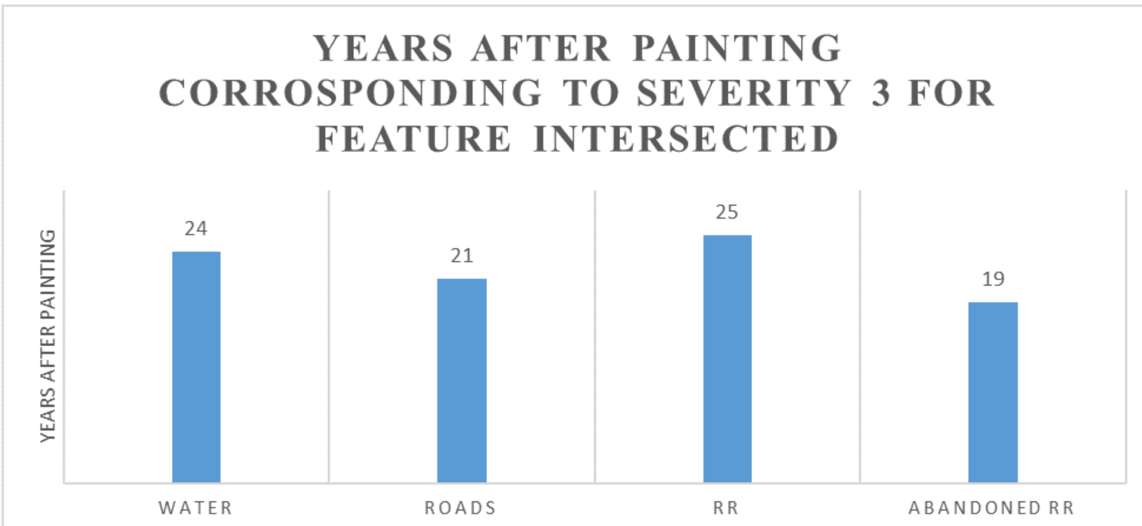

Fig. 4.29. Years after painting corresponding to severity rating 3 for feature intersected

The relationship between years after built and pack rust severity in splices

The Figure 4.30 plots the data for the severity rating of the pack rust in splices and the age of the bridge at the 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.

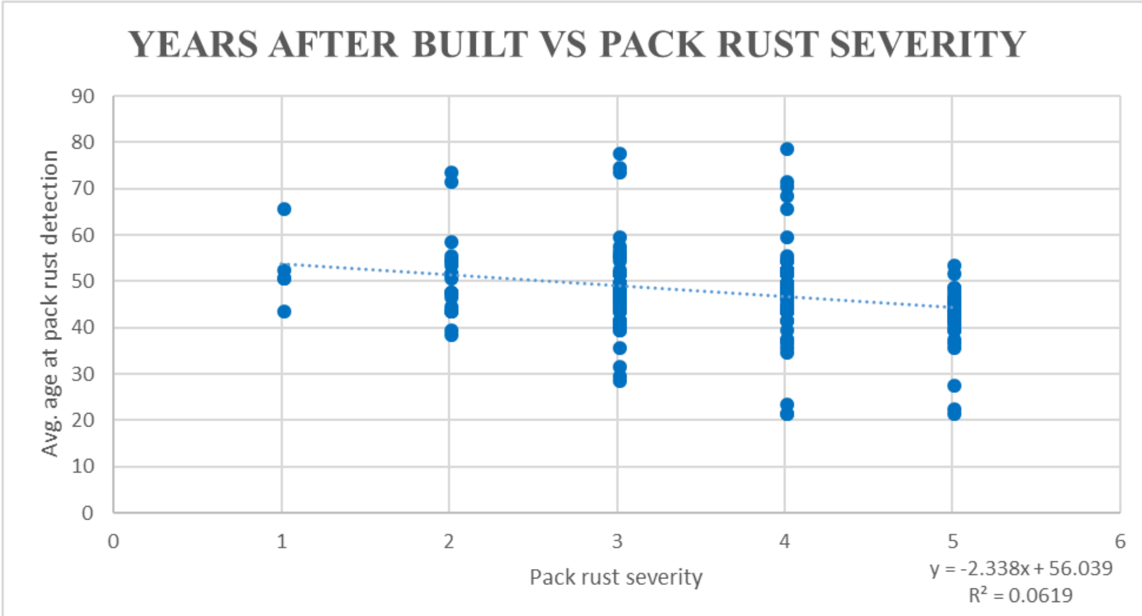

Fig. 4.30. Relation between years after built and pack rust severity in splices

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, under clearance, years after painting and the severity of the pack rust were compared to find a correlation. It took 23 years by a bridge that have under-clearance of 13 ft. to reach a condition of rating 3 and 22 years for a bridge with under-clearance of 25 ft. (22 years is the extrapolated value from the best-fit line Figure 4.28). 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 and greater	4	92	4.3

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 4.31 locates all the steel bridges in Indiana with pack rust (of any rating) in splices whose images are available. Almost all bridges that have pack rust in splices either carry interstate highway or are overpass bridges on interstate highways. There are some stretches of highways where there are no bridges with pack rust in splices. The distribution of bridges that have pack rust in splices based on the rating value is presented in Appendix Figure A.1

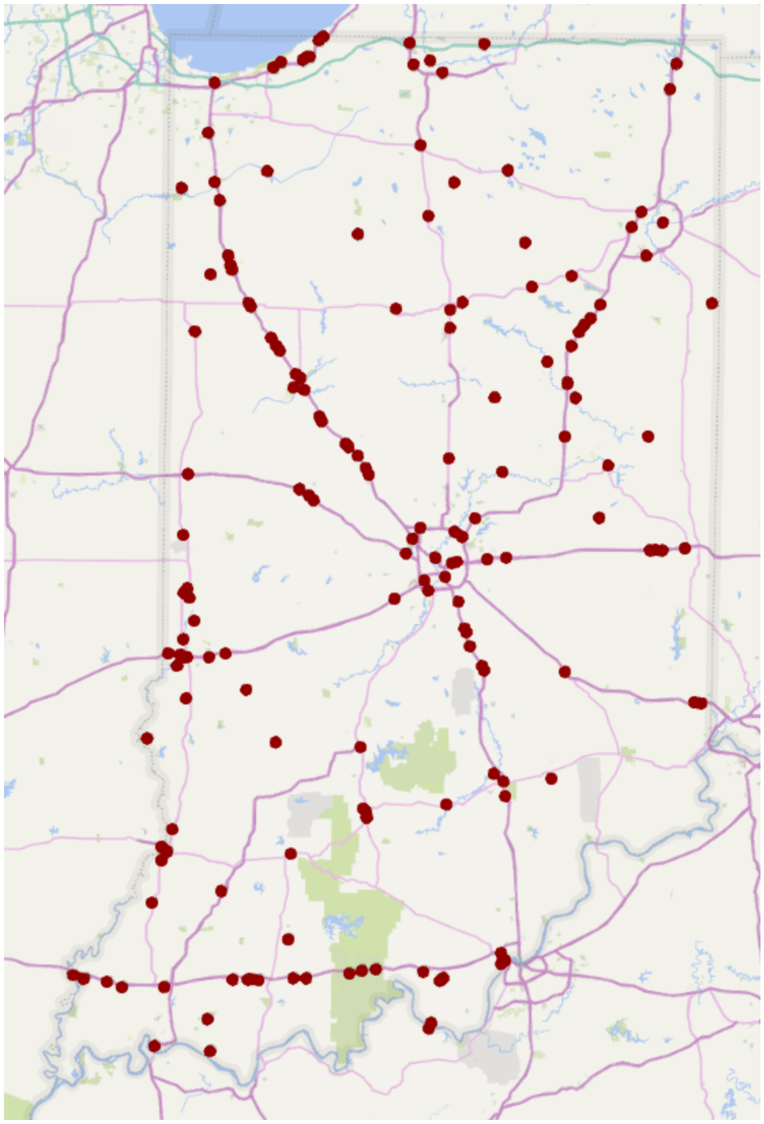

Fig. 4.31. Location of bridges with pack rust in splices

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

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. Figure 4.32 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 percent 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 R2 of about 80 percent compared to that for the splices which is less than 30 percent.

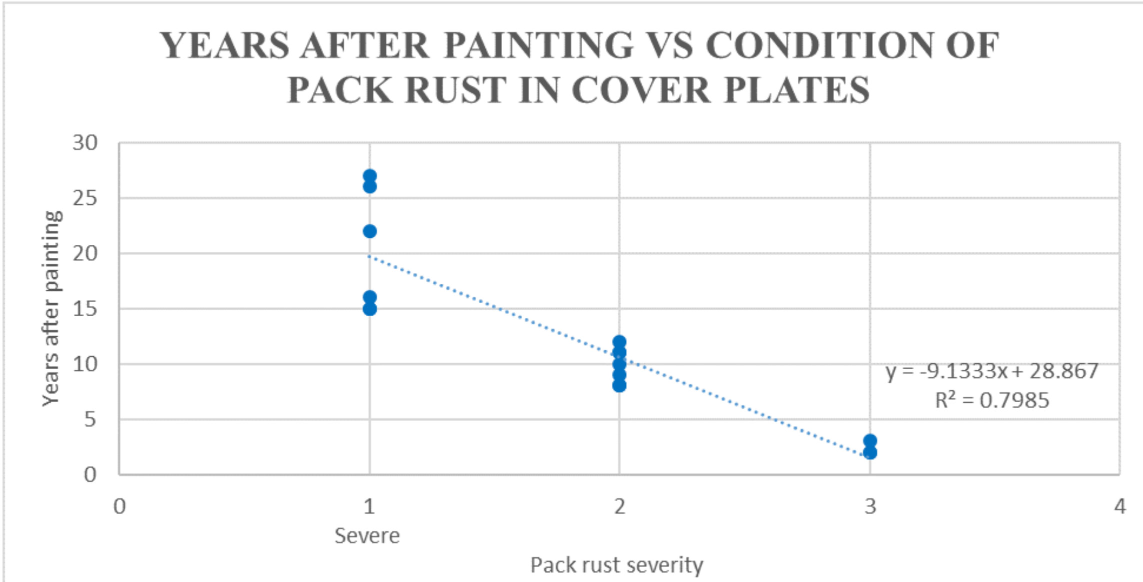

Fig. 4.32. Number of years after painting and the condition of pack rust in cover plates

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. FINITE ELEMENT ANALYSES

There are number of members in a bridge where pack rust formation is observed. Among all the members affected by pack rust, crevice corrosion in splice connections seems to be very critical. Three reasons which make splice connections to be analyzed in detail include: the large number of bridges with pack rust in splice connections; the difficulty of repair or replacement of the splice connections affected by pack rust; and the heavy damage caused by pack rust formation, which can result in some cases collapse of the bridge.

There are three possible locations where pack rust can form in splice connections. Those include corners of the splice plates, edges of the splice connection, and in the gap between the flange plates of the splice connection. The third case is the most common pack rust location among the three locations and is shown in Figure 5.1.

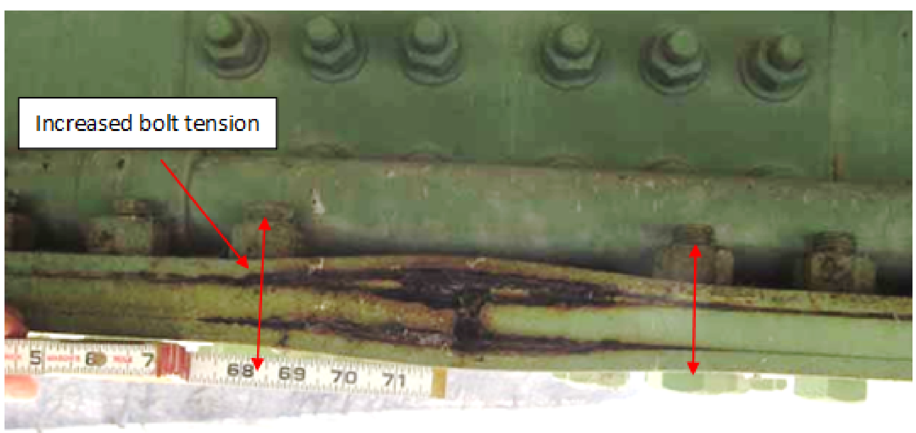

Fig. 5.1. Increase in the bolt tension due to bending of the splice plates

A 3D finite element model of a splice connection was modelled in ABAQUS to simulate the pressure exerted by pack rust on splice plates for the third case. The bending of the splice plates as shown in Figure 5.1 due to the pressure exerted by pack rust will lead to an increase in the tension in the adjacent bolts. Finite element analyses were conducted for the third case to find how much pack rust (the thickness of the pack rust) or how much bending of the splice plates will lead to failure of the adjacent bolts. The stresses and strains in the bolts are also checked for a 1-in splice plate deformation, since 1-in is the maximum deformation of splice plates observed in Indiana bridges.

The 3D model of connection was simplified to a beam member and analyzed in STAAD to compare a simplified model to the results from the ABAQUS.

5.1 Geometry of the connection

There are two splice plates of dimensions 4 in. x 9 in. x 0.5 in. and two flange plates of dimensions 5.5 in. x 4 in. x 1 in. Two splice connections were modeled: one with 1-in diameter bolts and second with 7/8-in diameter bolts. The edge distance and the end distance of the bolts for the flange plates and splice plates and the bolt hole diameter for 1-in diameter bolt and 7/8-in diameter bolts and are shown in Figure 5.2 and Figure 5.5 respectively. The assembly of the splice connection for 1-in diameter bolt and 7/8-in diameter bolt are shown in Figure 5.3 and Figure 5.6. The dimension of the bolt and the bolt hole was referenced from the Specification for Structural Joints Using ASTM A325 or A490 Bolts, (2004) [19]. The splice connection in bridges generally have a gap of 1/4-in between the flange plates as shown in the Figure 5.4. This gap is essential for pack rust formation in the actual splice connection. This gap is the primary point of entry for the moisture into the splice connection. In the numerical model, the bending of the splice plate due to the pressure exerted by pack rust is only required. The bending of the splice plate is independent of the presence of the gap between the flange plates in the numerical model.

The particular sizing and geometry of the splice connection is typical for the Indiana Bridges.

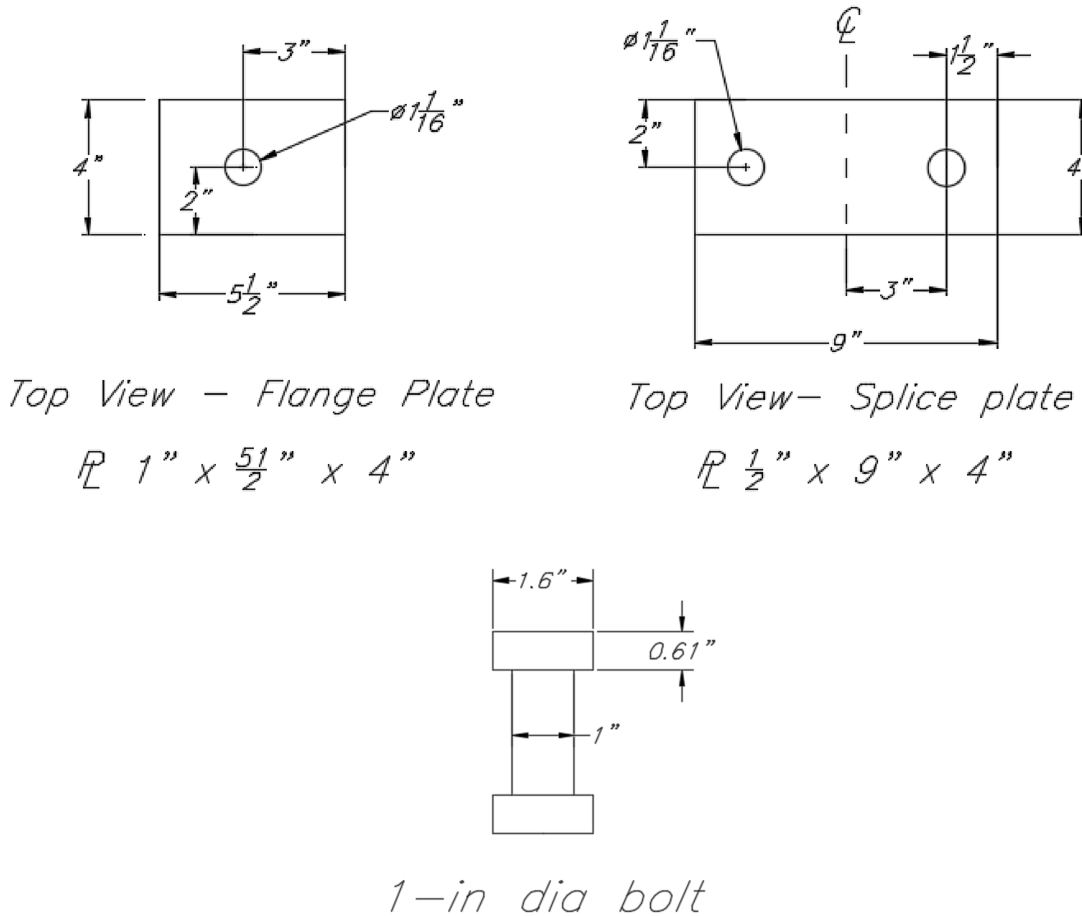

Fig. 5.2. Parts of the splice connection with 1-in diameter bolts (Splice plates, flange plates, and bolts)

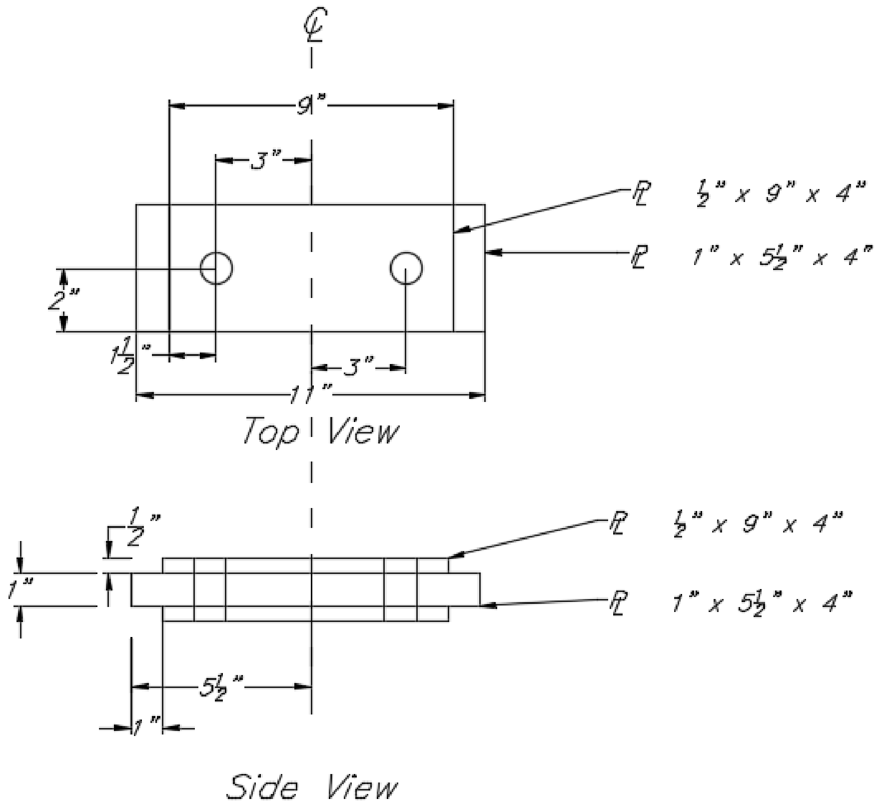

Fig. 5.3. Assembly of the splice connection with 1-in diameter bolts

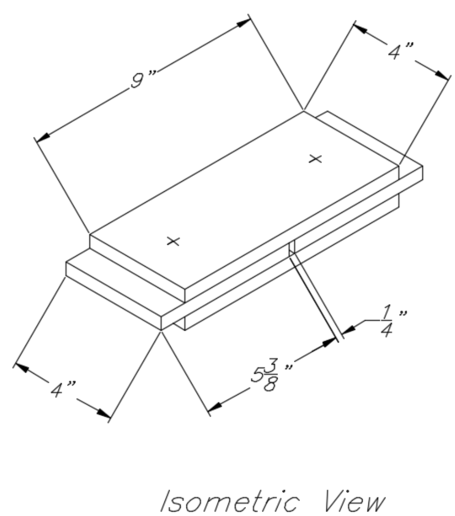

Fig. 5.4. Isometric view of the splice connection with a gap between the flange pl-
ates

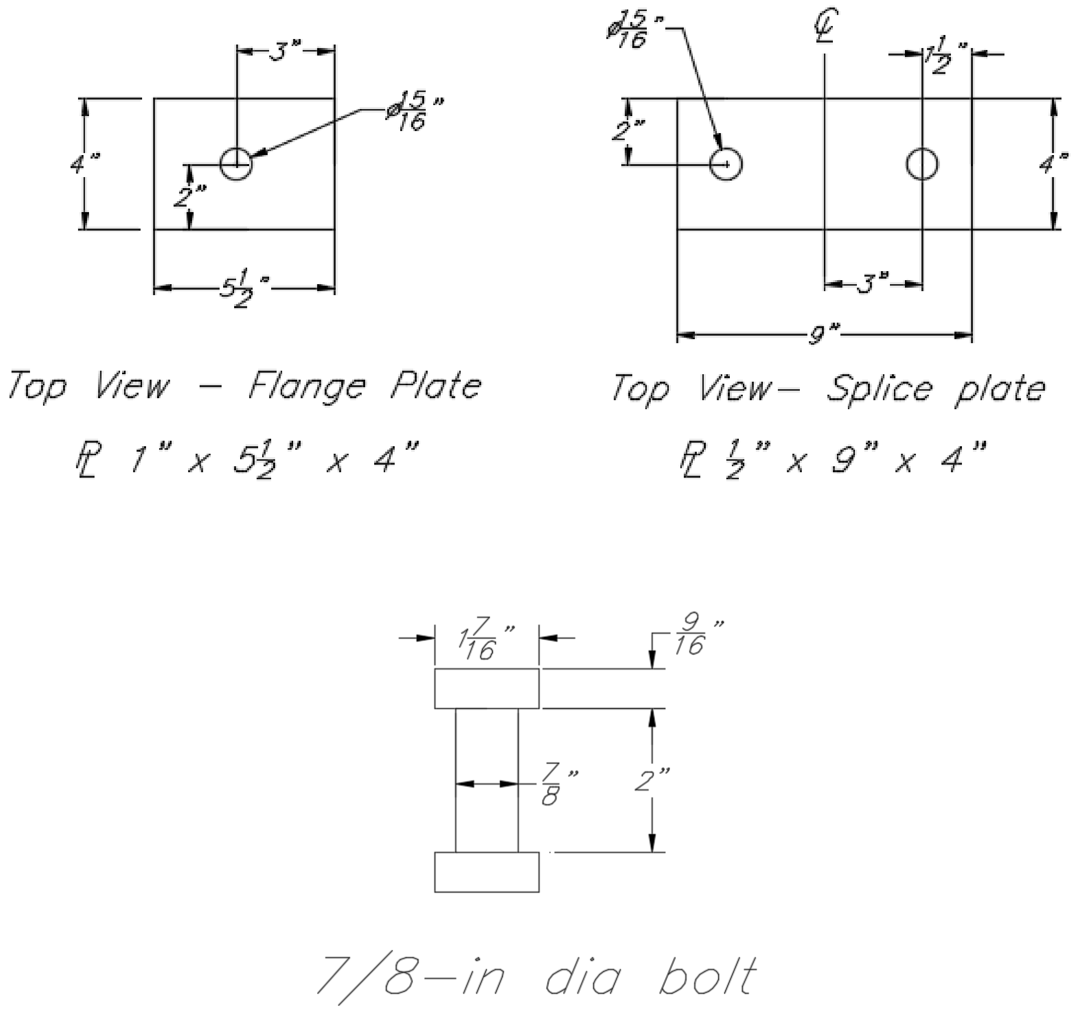

Fig. 5.5. Parts of the splice connection with 7/8-in diameter bolts (Splice plates, flange plates, and bolts)

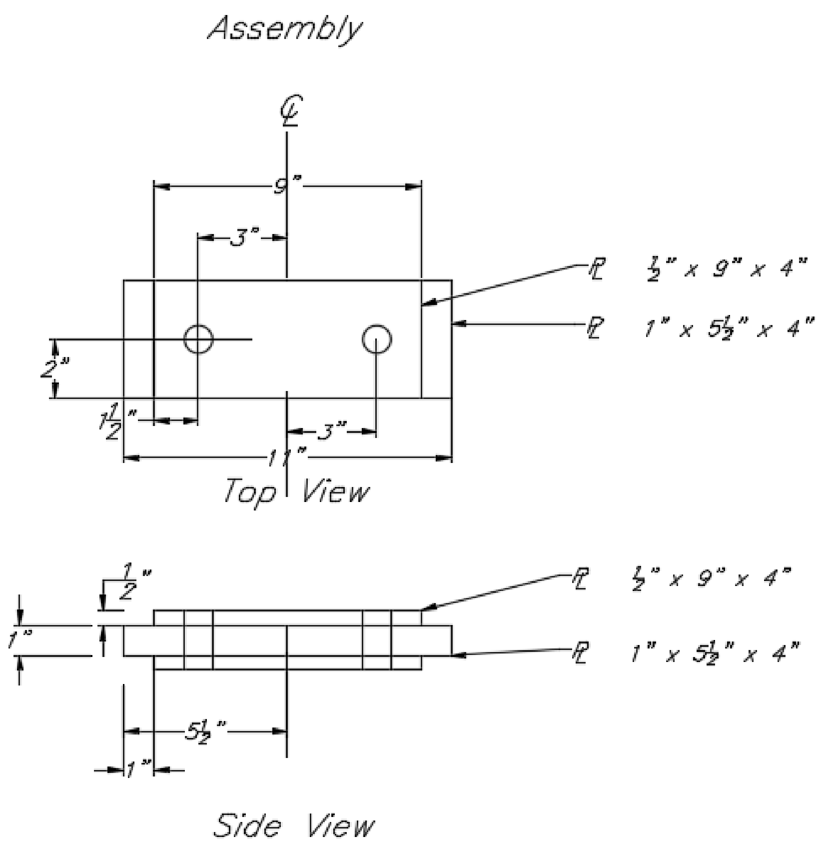

Fig. 5.6. Assembly of the splice connection with 7/8-in diameter bolts

5.1.1 ABAQUS Model

Parts

The components of splice connection including bolts, splice plates and flange plates are modeled as 3D and deformable parts.

Material property

Material properties are simplified for the use in the software. ASTM A325 bolts have a peak ultimate strength of 120 ksi, which decreases to 100 ksi at fracture. Revised values for plastic stress vs plastic strains are tabulated in Table 5.1 along with the elastic property for the ASTM A325 bolts. Negative stiffness between peak ultimate strength and the fracture strength is not considered. Perfectly plastic behavior is considered between strain of 0.062-in/in and 0.16-in/in.

Material properties for the ASTM A572 grade 50 steel are tabulated in Table 5.2. The behavior of the material is elastic-perfectly plastic.

Table 5.1.
The material property used for the ASTM A325 bolts

Elastic Property	
Modulus of Elasticity (psi)	29,000,000
Poisson's Ratio	0.3

Plastic Property	
Plastic Stress (psi)	Plastic Strain
92,000	0
95,000	0.008
100,000	0.062
100,000	0.16

Table 5.2.
Material property used for the ASTM A572 Grade 50 steel

Elastic Property	
Modulus of Elasticity (psi)	29,000,000
Poisson'Ratio	0.3

Plastic Property	
Plastic Stress (psi)	Plastic Strain
50,000	0
60,000	0.16

Steps

Each analysis is executed in two steps: pretension and rust pressure. The nonlinear geometry (Nlgeom) setting is set off for this model. A model with nonlinear geometry when set on, the analysis time was 7 hrs to reach half the total load, so further analyses were not conducted. The automatic stabilization was used with default energy dissipation fraction of 0.002 and maximum ratio of stabilization to strain energy of 0.05 was used for both the steps.

Interaction

The interaction between the surfaces in contact are assigned to be frictionless to reduce complexities and computation time. The forces applied in the model will be in the direction normal to the surface and hence the sliding of the parts against each other is not very significant.

The frictionless behavior is an assumption in the model. Splice connections are designed as slip critical connection. The assumption is discussed in later section.

Loads and boundary condition

Bolt load

The 1-in diameter bolts are pre-tensioned to 51 kips and 7/8-in diameter bolts are pre-tensioned to 39 kips as per Table 8.1 in Specification for Structural Joints Using ASTM A325 or A490 Bolts, (2004) [19]. The pretension was applied using the bolt load function in the ABAQUS. The bolt load is applied in a single step.

Pack rust load

The region over which the pack rust exists inside the crevice, between the splice plate and the flange plates is unknown. The pressure applied by pack rust and the variation of the pressure with respect to the depth inside the crevice is also unknown. The only parameter known is the deflection of the splice plates due to pack rust as observed from the inspection reports of the bridges in Indiana. In this study the process is to apply loads over a specific region such that the bending of the splice plates causes over-stressing in the bolts and cause them to fail.

The region over which pack rust exists is assumed to be a semi-circular region as shown in the Figure 5.7. The radius of the semi-circular region will increase with time as the pack rust severity increases. Considering the extreme case, the radius of the region is taken to be 3 inches. The load applied is a uniform maximum pressure of 4000 psi over the marked region in a single step.

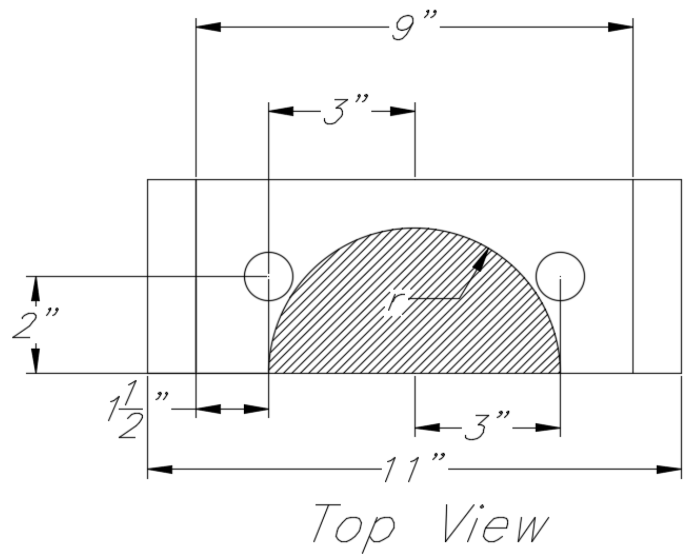

Fig. 5.7. Region over which pack rust pressure is applied

The splice connection is not subjected to any shear forces, which exists in the actual splice connections. This is done to reduce the complexities in the model. It is acknowledged that the strength of the bolts in tension reduces with the introduction of shear forces on the bolts.

The ends of the flange plates were fixed with respect to position and rotation as shown in Figure 5.8.

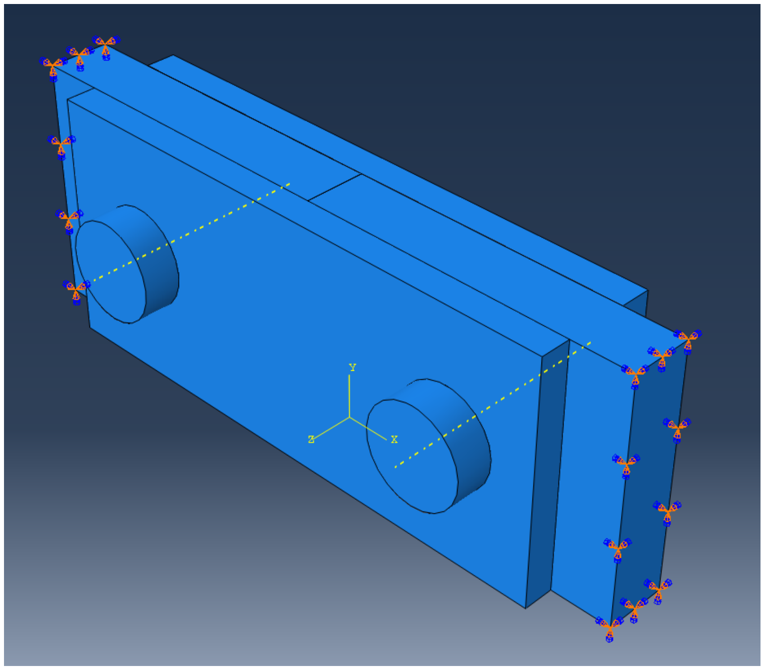

Fig. 5.8. Boundary condition applied in the model

Mesh

The mesh element used is C3D8R i.e. hex type, linear geometric order with reduced integration. The approximate global size of the element used in the bolt and the splice plate is 0.1-in and for the flange plate it is 0.2-in. Total number of nodes on the model are 69,634 and total number of elements are 55,892. Figure 5.9 shows the meshed model of the splice connection in ABAQUS. It is seen that the mesh is finer for the splice plates and bolts and a coarse mesh is used for flange plates since the behavior of the flange plate is not important in this study.

Assumptions

The ABAQUS model is based on three assumptions. Firstly, the pack rust formation and crevice corrosion will corrode the bolt shank also and will reduce the cross section of the bolts. The reduction of cross section will reduce the tensile capacity

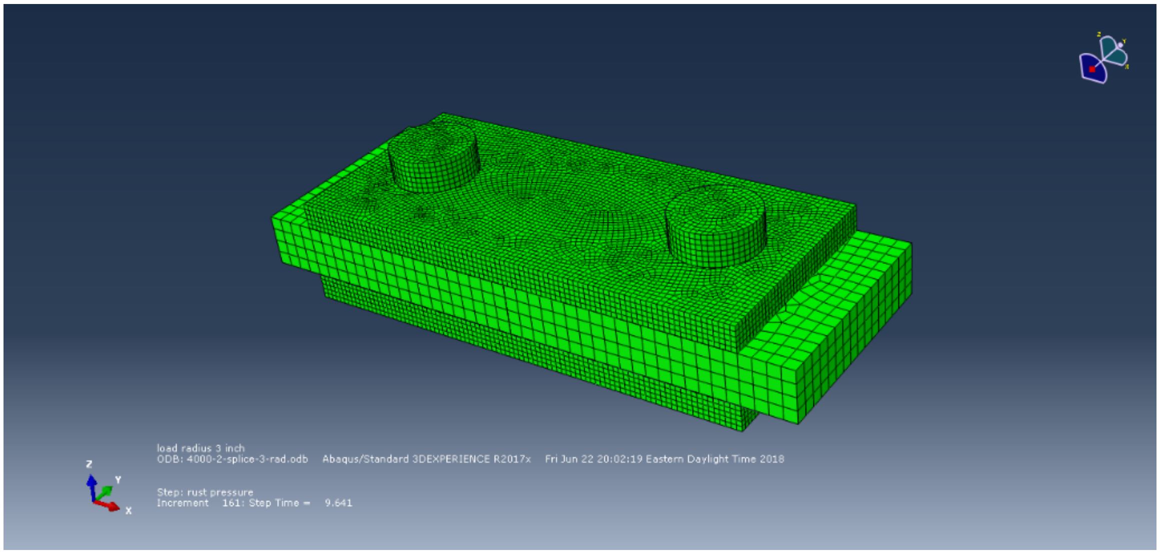

Fig. 5.9. Region over which pack rust pressure is applied

of the bolts. In the current study reduction in the cross section of the bolts is not considered. Second, shear forces are not applied to the flange plates. The bolts in the splice connection will be under tension from pack rust and shear forces. The tensile strength of the bolts will reduce under the dual action of tensile and shearing forces. Third, the surfaces in contact in the model are modeled as frictionless. In actuality the splice connections are designed as slip critical. As the model only considers the forces due to pack rust and no shearing action between the plates friction will not play a major role in the behavior of the splice plates.

Results

Splice connection with 1-in diameter bolts

For the 1-in diameter bolts the analyses aborted at 3,850 psi pressure with the maximum deflection in the splice plate of 5.9 inch at the edge. The scaled deformed shape of the splice connection with 1-in diameter bolts is shown in the Figure 5.10.

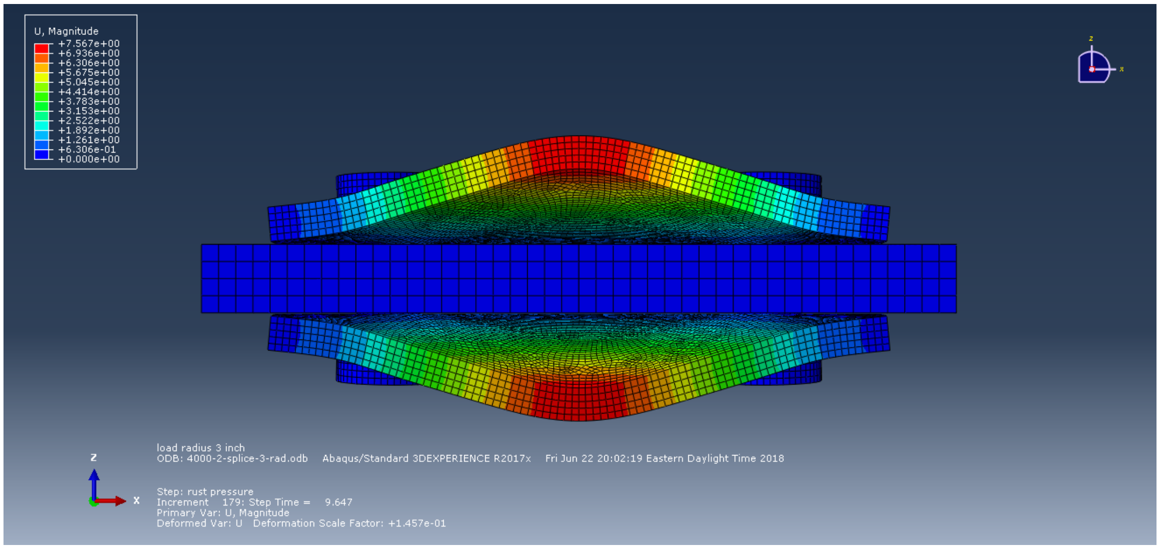

Fig. 5.10. Deformed shape of the splice connection

At this load the bending of the splice plate leads to maximum principal strain of 0.17-in/in in the bolt shank thus reaching the strain at the point of failure. It is seen from Figure 5.11 that most of the bolt shank has yielded and the bottom of the bolt head has also yielded. It is also observed that the bolt not only experiences elongation but also bending which is expected due to the nature of loading. Figure 5.11 is plotted with a scaling factor of 1.

The observed maximum deflection of splice plates in the state of Indiana due to pack rust are in range from 3/4-in. to 1-in. Splice connections with such deformations due to pack rust are rated 1, which is the most severe pack rust severity rating used in this study. There were only three to four bridges with such a severe pack rust condition detected in splice connections. The bolts in the splice connection of the observed severe cases of pack rust have not failed at such deformations.

The ABAQUS analyses shows that the maximum principal strain in the bolt shank at 1-in of splice plate deformation is 0.0075-in/in and part of the bolt shank has yielded as shown in the Figure 5.12. A uniform pressure of 2,960 psi over a semi-circular region of radius 3 inches causes 1-in of deformation in splice plate.

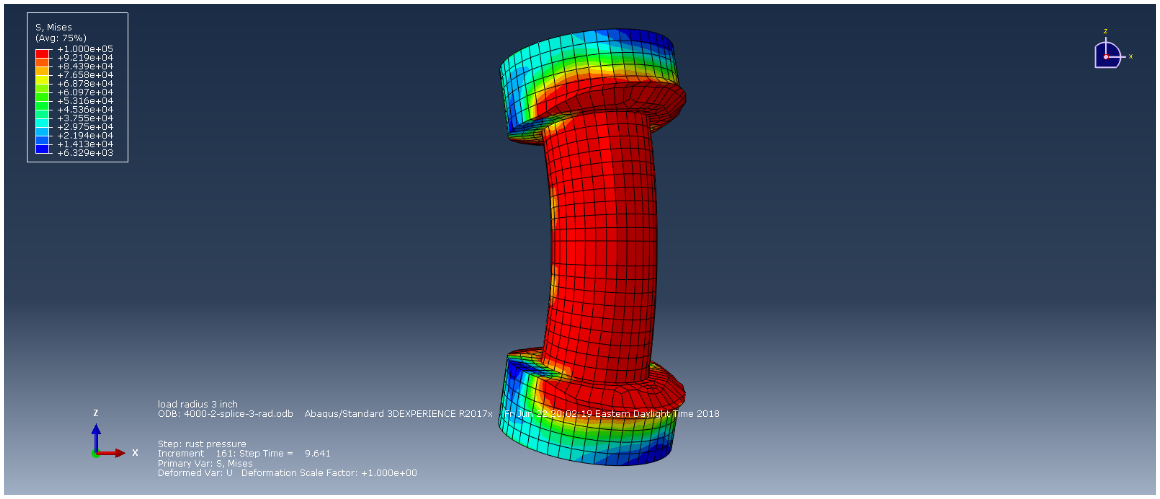

Fig. 5.11. Mises stress in bolt at 3,850 psi load, (5.9-in splice deformation)

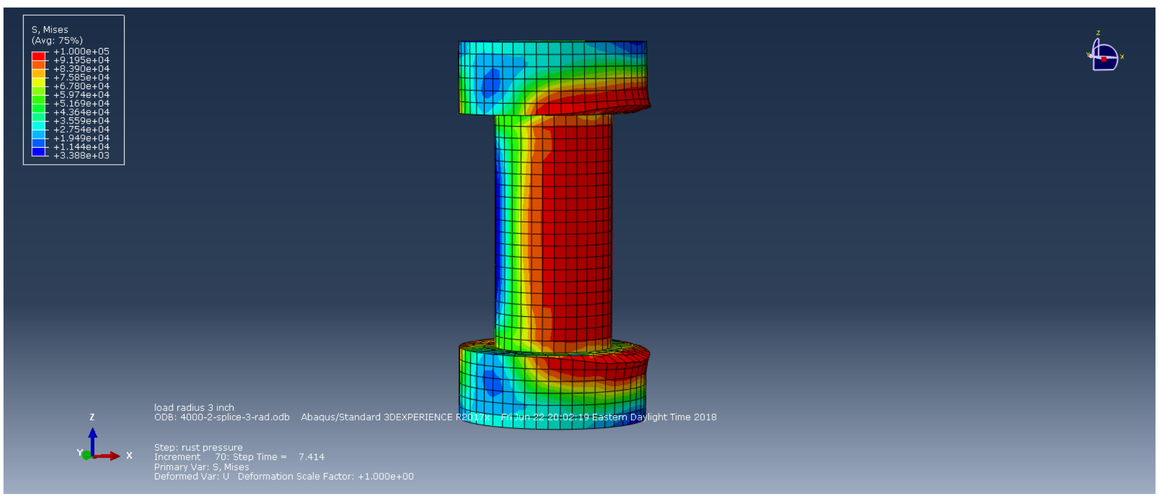

Fig. 5.12. Mises stress in bolt at 2,960 psi, (1-in. splice deformation)

Figure 5.13 shows the maximum principal strain in the splice plate. It is observed that the strain at the edge of the contact between the bolt head and the splice plates marked as 2 in Figure 5.13 are large, and the maximum principal strain reached is 1.68 in splice plate. The value of strain at location 1 marked in Figure 5.13 is 0.38.

Figure 5.14 shows the Mises stress in the splice plate. It is seen that almost the entire plate has yielded. The splice plates in the current analyses did not fail.

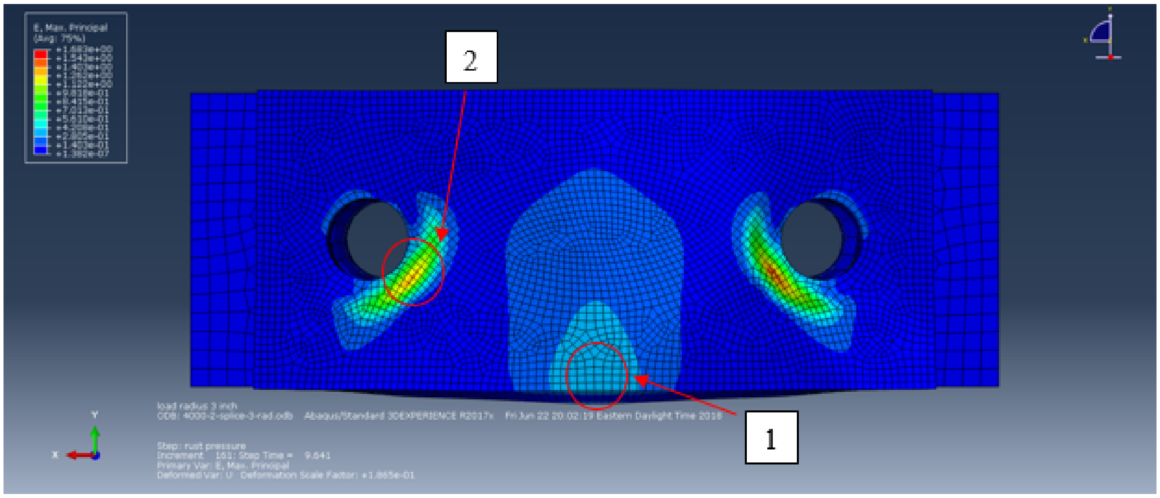

Fig. 5.13. Maximum principal strain in splice plate at failure of bolts

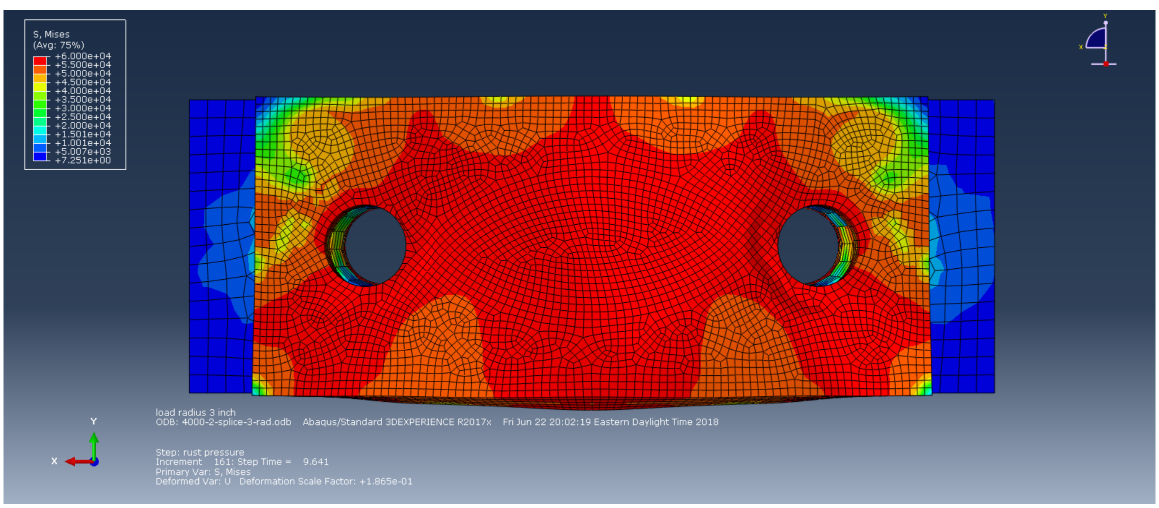

Fig. 5.14. Mises stress in the splice plate at failure of bolts

Splice connection with 7/8-in diameter bolts

For the 7/8-in diameter bolts, the analyses aborted at 3,100 psi. The maximum splice plate deformation observed at this pressure was 3.8-in. At this pressure the maximum principal tensile strain in bolt shank is observed to be 0.19-in/in.

The splice plates deforms by 1-in when a uniform pressure of 2,500 psi is applied over a semi-circular region of radius 3-in. The maximum principal tensile strains in the bolt shank at this deformation is 0.0138-in/in.

Discussion

Table 5.3 summarizes pack rust pressure, strain in bolt shank and splice plate deformation for the splice connection with 7/8-in diameter bolts and 1-in diameter bolts. As expected, the pressure to fracture 7/8-in diameter bolts is less than to fracture 1-in diameter bolts.

The pressure at which the bolts fail in the ABAQUS model are considered to be larger than the pressure to fracture the bolts in reality. This is because of the assumptions made in the model. During the crevice corrosion process there will be reduction in the cross-section of the bolts and the bolts will also experience shear forces. Both of these factors are not considered in the analyses. Thus, in actuality the tensile strength of the bolts will be reduced. The pressure to fracture the bolts will also be less than what is observed in current study.

The most severe cases of pack rust in splice connection observed in the state of Indiana have a splice plate deformation of 1-in. From the analyses, the stress and strains seems to be in safe limits for a 1-in deformation of splice plates. If the assumptions are not made the strains in the bolt shank would be higher than what is observed in this study for the splice deformation of 1-in. But, based on the field data, none of the four splices with such deformation have experienced bolt failure. The bolt strength in this study is also chosen to be on the conservative side. The tensile strength of the A325 bolts is 120 ksi but in this study the tensile strength is taken as 100 ksi. The strains in the 1-in diameter bolt shank at 1-in deformation is also 4.6 percent of the strain at fracture. Based on the above assumptions in the analyses and field observation, it can be concluded that the bolts are safe at 1-in of splice plate deformation.

Table 5.3.
ABAQUS results summary

	7/8-in dia bolt		1-in dia bolt	
	at failure	at 1-in def.	at failure	at 1-in def.
P (psi)	3,100	2,500	3,850	2,960
Strain in bolt shank (in/in)	0.19	0.014	0.17	0.0075
Splice deformation (in)	3.8	1.0	5.9	1.0

5.1.2 Simplified model of splice connection with 1-in diameter bolts using STAAD Pro

A simplified model of the splice connection with 1-in diameter bolts was modeled in STAAD Pro. A shaded strip of the plate between the bolts is modeled as a beam element as shown in the Figure 5.15. A beam of width 0.1-in (the mesh size of splice plate in ABAQUS model), 1-in and 2-in with 0.5-in of depth (thickness of splice plate) was modeled. The length of the beam is 5-in. The material property of the beam is same as that of the steel plate in ABAQUS model (Yield stress = 50 ksi and Ultimate stress = 60 ksi).

The beam ends are restrained against rotation in one case and free to rotate in other case. Movevent in the direction normal to the splice plate is modeled as a spring support. The stiffness of the spring is based on the stiffness of the bolts.

The beam model does not seem to be an appropriate simplification for a plate model to calculate deflection with a non-symmetrical loading region.

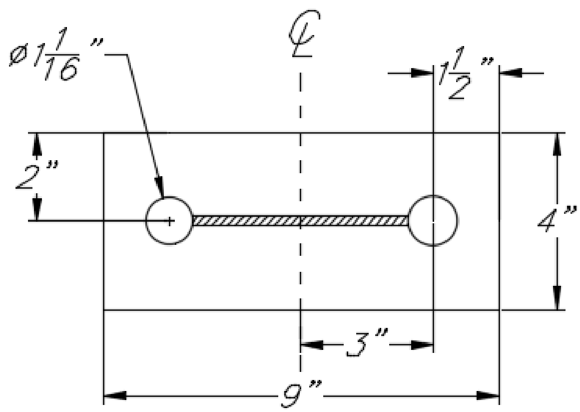

Fig. 5.15. Simplified beam element

5.1.3 Splice plate deformation based on theory of plates

Timoshenko and Woinowsky-Krieger [20] provides an equation for deflection of uniformly loaded rectangular plates with three edges simply supported and the fourth edge being free. The generalized equation for the deflection is shown in Equation 5.1 and the formula for D is presented in Equation 5.2. The equation by Timoshenko calculates the elastic deformation in a rectangular plate.

$$\delta = \alpha \frac{qa^4}{D} \tag{5.1}$$

$$D = \frac{Eh^4}{12(1-\nu^2)} \tag{5.2}$$

where,

α = coefficient dependent on the aspect ratio of the rectangular plate

q = uniform pressure exerted on the plate

a = width of the free edge of rectangular plate

E = Young's Modulus of the rectangular plate

h = thickness of the rectangular plate

ν = poisson's ratio (0.3)

The deformation of the splice plate due to pack rust can be simplified to a uniformly loaded rectangular plate with three edges simply supported and the fourth edge free. Proper dimension of the rectangular plate needs to be assumed to find elastic deflection in the plate. The ABAQUS model includes both elastic and plastic deformation whereas the equation by Timoshenko only calculates the elastic deformation.

The ABAQUS model shows that the splice plates starts to yield near the bolt region and at the mid-length of the splices when the deflection at the edge is just 0.052-in. The pressure required to cause this deflection and yielding condition is 1800 psi. The red colored elements in the ABAQUS model shown in Figure 5.16 represents the region of the plate that has already yielded and blue colored elements have stresses that are below yield stress. The equivalent rectangular plate size of 6-in by 3-in and plate thickness of 0.5-in is used to calculate the elastic deflection using the equation by Timoshenko. In the assumed dimension of the rectangular plate, the length of the free edge is 6-in and width is 3-in. For this plate size the value of coefficient α is 0.0071. The calculated maximum deflection at the free edge using Equation 5.3 is 0.05-in. The difference in the deflection value can be explained by the regions of the splice plate undergoing plastic deformation leading to increased splice plate deformation.

The pressure in the ABAQUS model is uniformly distributed over a semi-circular region with radius of 3-in. In the Timoshenko model the pressure is uniformly distributed over the entire plate. The deflection in splice plate at a pressure of 1800 psi is shown in Figure 5.17. The figure shows two red circles marked at each end of the splice plate, denoting that these edges of the splice plates are still in contact with the flange plates. The splice plate is not in contact with the flange plate for approximately 6-in at the edge and 3-in perpendicular to the edge as marked in Figure 5.17. Therefore, the approximation of the plate dimension and deflection for the Timoshenko model is in good comparison with the ABAQUS model because most of the splice plate remains elastic.

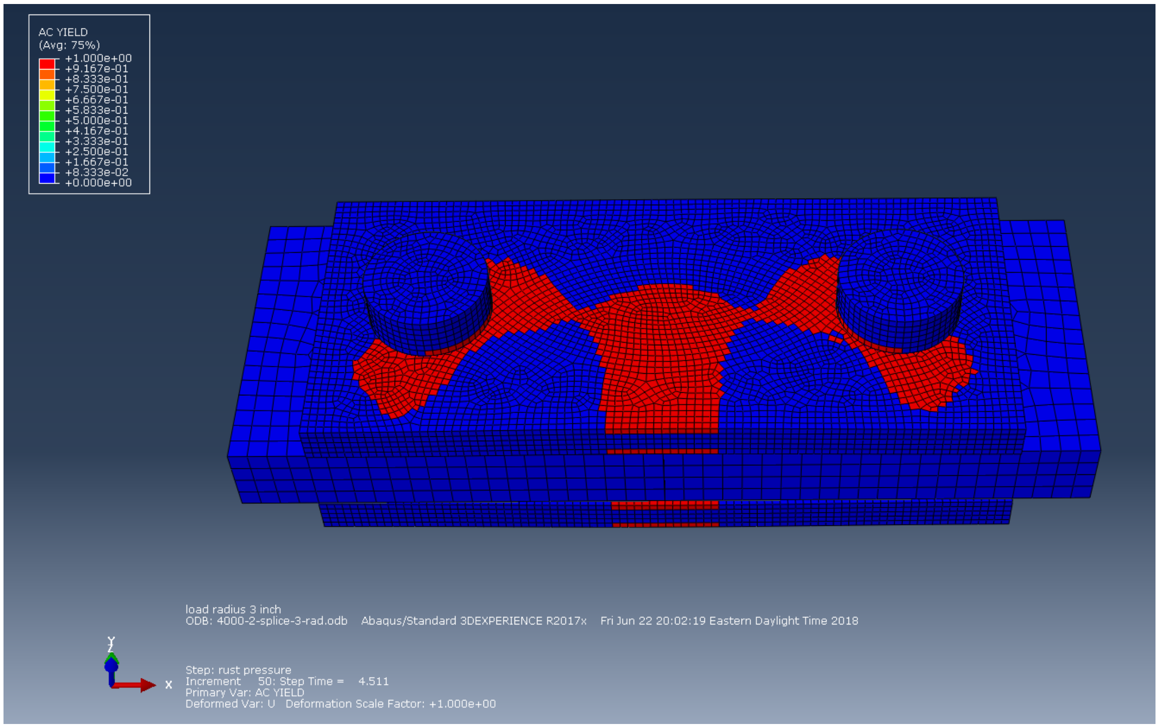

Fig. 5.16. Regions of the splice connection that has yielded at pressure of 1800 psi (elements in red color)

$$\delta = 0.0071 \frac{qa^4}{D} \tag{5.3}$$

When the deformation in the splice plates reaches 1-in at a pressure of 2960 psi, entire splice plate has undergone yielding, as shown in Figure 5.18. The elements in the red color are the elements that have stresses beyond yield stress and elements in blue color are the elements which have stresses below the yield stress. The corners of the splice plates have also displaced, they are no more in contact with the flange plates, as shown in Figure 5.19. This observation from ABAQUS supports the assumption of a 10-in free edge of the rectangular plate with the width of the plate being 4-in for the Timoshenko model. The value of the coefficient α in Equation 5.1 for the plate aspect ratio of 0.4 is 0.00584. Using the equation by Timoshenko the calculated maximum deflection in the plate is 0.52-in. The deformation in the ABAQUS model is double

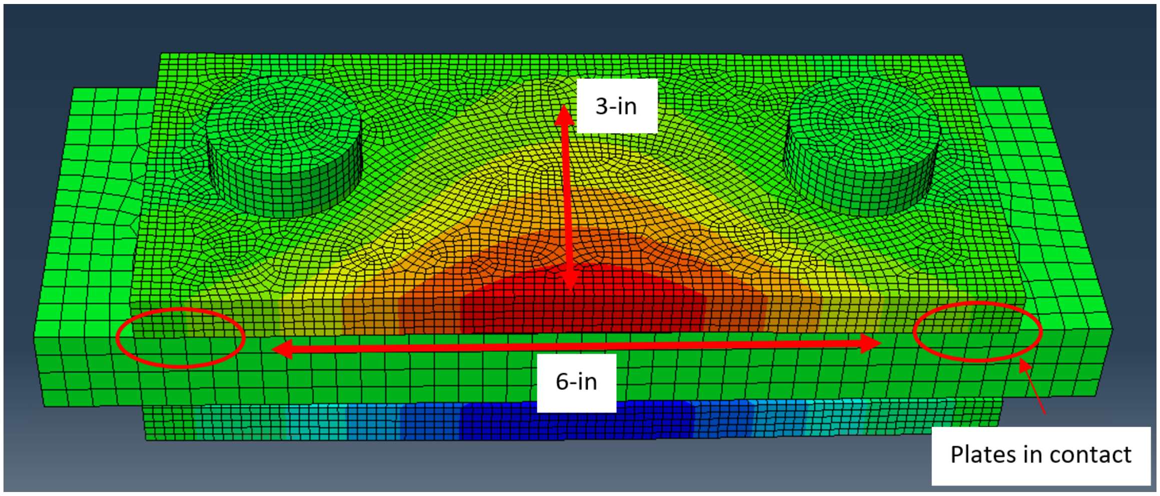

Fig. 5.17. Splice plate deflection at 1800 psi pressure

the deformation calculated using Equation 5.4 for a pressure of 2960 psi even though the total force acting on the rectangular plate model by Timoshenko is more than the total force acting on the ABAQUS model. Reasons for this observation includes: first, the ABAQUS splice connection model allows for the plastic deformation and the equation by Timoshenko only considers elastic deformation in the plate and second, the bolts in the ABAQUS model also deforms thus allowing increased deformation in the splice plate whereas, the boundary condition in the Timoshenko's elastic model is fixed against transitional movement.

$$\delta = 0.00584 \frac{qa^4}{D} \tag{5.4}$$

Table 5.4 compares the deformation of splice plate from ABAQUS analyses and from the equation provided by Timoshenko. For small deformation in splice plate i.e. plates have not yielded, both finite element analyses and the Timoshenko equation obtain comparative deformations. When the deformation in splice plate increases, the difference in deformation calculated using both the approaches also increases. It

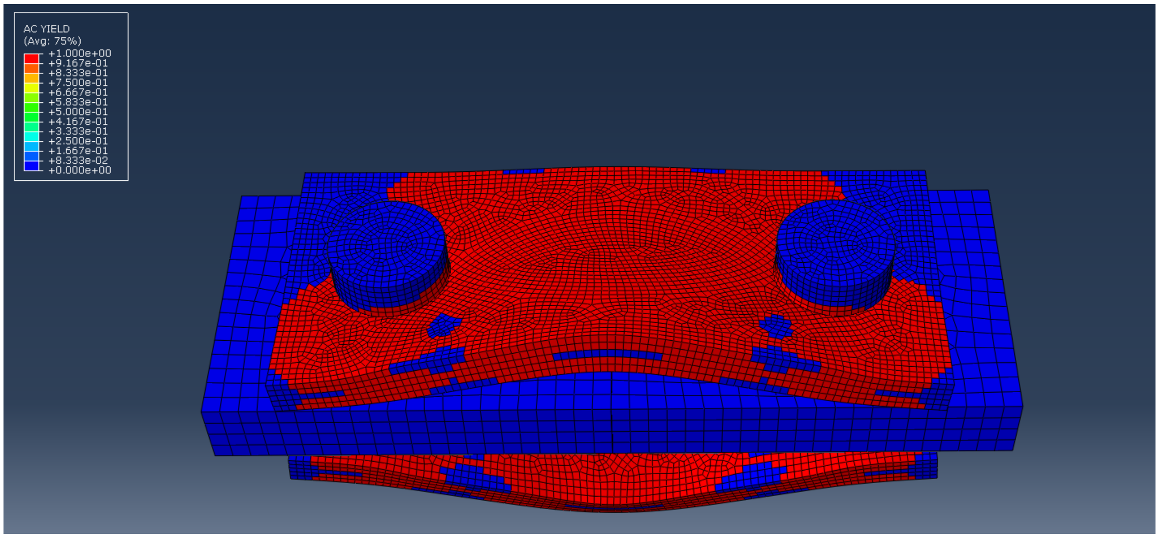

Fig. 5.18. Yielded region of splice plate at 1-in deformation

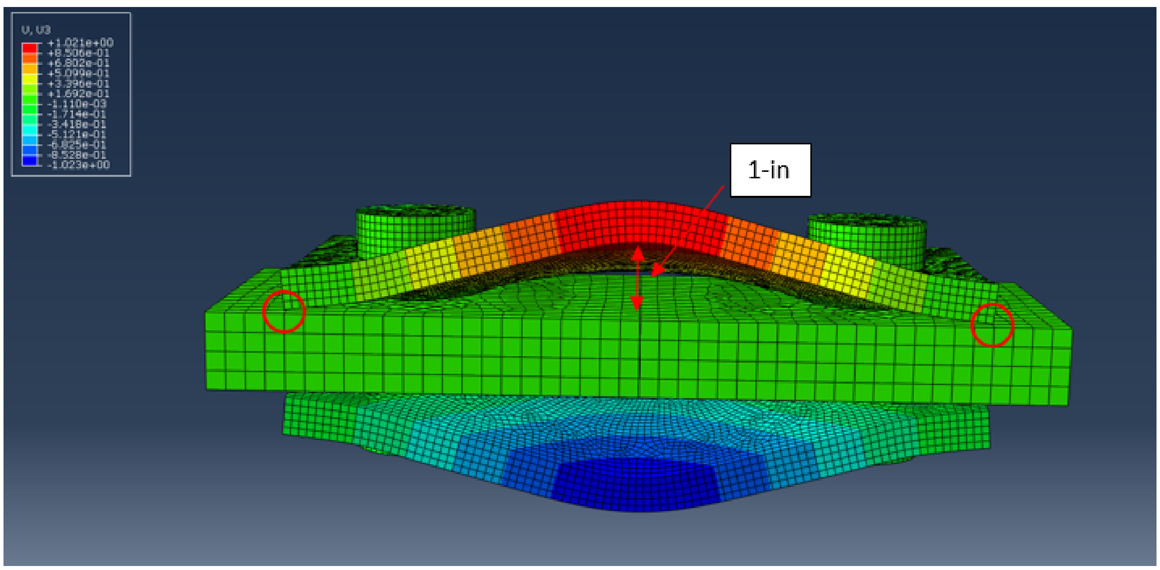

Fig. 5.19. 1-in of splice plate deflection at 2960 psi pressure

is important to note that even 0.052-in deformation in splice plate causes splice plates to yield to some degree.

Table 5.4.
Results summary from both approaches

Splice plate deformation (in)		Timoshenko Plate dimension	Pressure (ksi)
ABAQUS	Timoshenko Equation		
0.052	0.05	6-in x 3-in	1.8
1.0	0.52	10-in x 4-in	2.96

6. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 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 percent of the bridges with such details. Moreover, pack rust occurrence was less than 10 percent in end diaphragms, beam cover plates and cross bracings. The pack rust occurrence in rocker bearings was found to be around 30 percent, and for splice plate connections it was 13 percent. The pack rust occurrence in bearings (30 percent) and splice connection (13 percent) is less compared to gusset plate and hinge-pin connections (greater than 90 percent), 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 percent and railroads by 11 percent. 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 distance and lower 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).

6.2 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.

Further study should be done to investigate the effectiveness of stripe coating and 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. In case, if rust bleeding is observed in splice connection, use of 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 may 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.

Finite element analyses of splice connections indicate that 1-in of splice plate deformation will not lead to failure of the adjacent bolts with diameters of 1-in and 7/8-in. At such deformations part of the bolt yields. One inch of splice plate deformation creates a maximum principal strain of 0.0075-in/in in 1-in diameter bolt shanks and 0.014-in/in in 7/8-in diameter bolts, which is well below fracture strain of 0.16. For the bolts to fracture the required splice plate deformation is 3.8-in and 5.9-in for 7/8-in diameter bolts and 1-in diameter bolts, respectively. Splice plate deformation of just 0.05-in leads to yielding in the splice plates near the bolts.

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APPENDICES

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" & <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	

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.

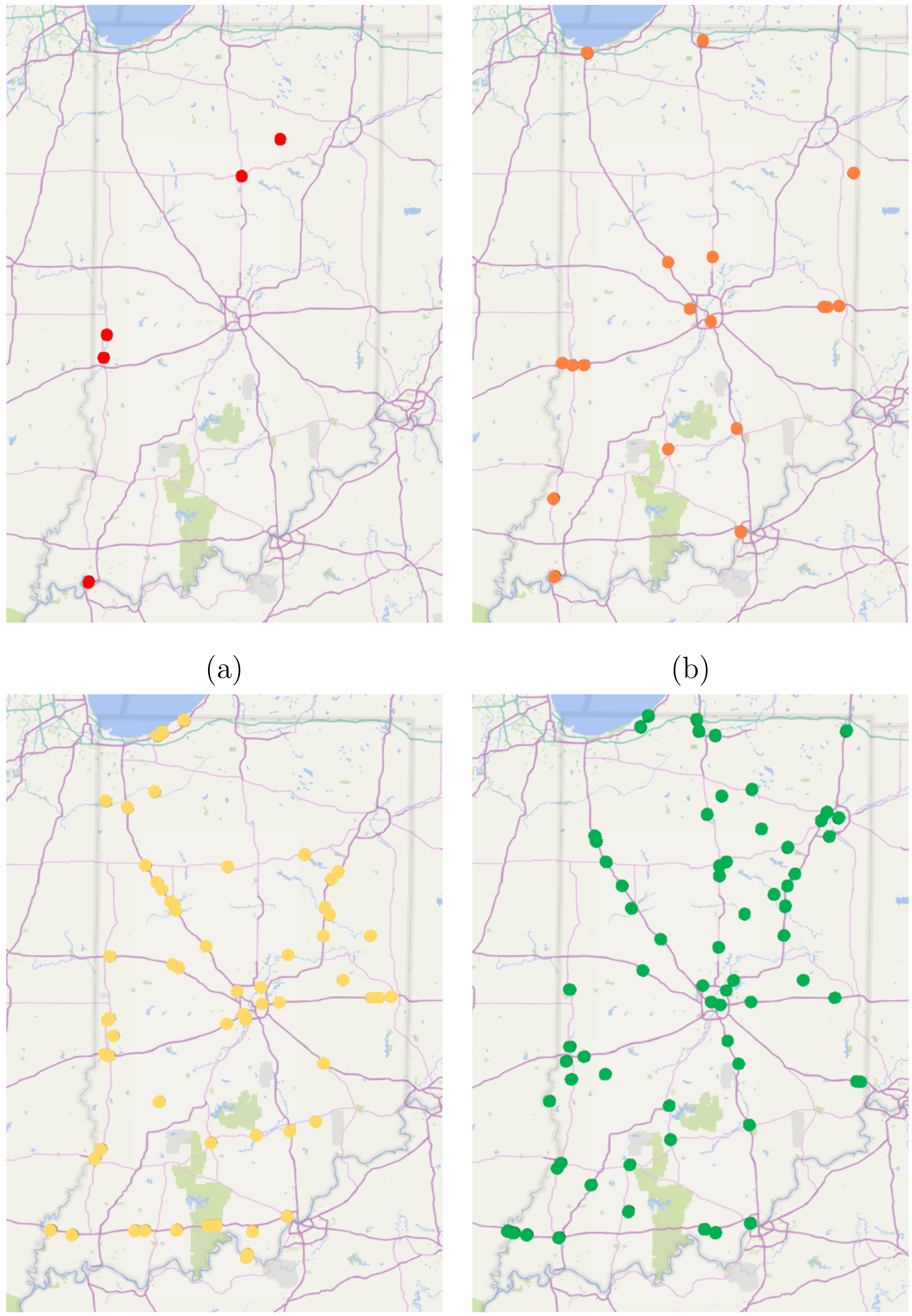
(Notation: pc primer coat, spc stripe coat of primer coat, ic intermediate coat, sic stripe coat of intermediate coat, fc finish coat, sfc stripe coat of finish coat)

Table A.2.
Breakdown of the number of bridges that intersects particular feature

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

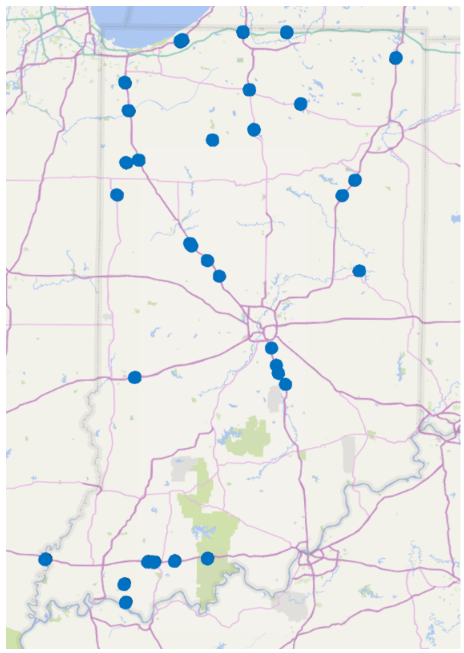
Table A.3.
Number of bridges with pack rust in splice connections over given intersecting feature

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
La Porte					
	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



(a) (b)

(c) (d)



(e)

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