BEHAVIOR OF BOLTED CONNECTIONS IN RAILROAD DIAMOND CROSSINGS

BY

MARTIN V. WHITE

THESIS

Submitted in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering in the Graduate College of the University of Illinois at Urbana-Champaign, 2012

Urbana, Illinois

Adviser:

Assistant Professor Larry Fahnestock
Abstract

Maintenance and replacement costs of special track work elements are an expensive problem for the railroad industry. In the railroad industry, special track work refers to special railroad elements, including turnouts, switches, and diamond crossings, that are used where rails join or cross. The repeated impact and dynamic loads that diamond crossings are subjected to result in alignment problems and more rapid deterioration of the special track work elements compared to open track. This research project aims to perform finite element analysis of a diamond crossing frog in order to observe the behavior of diamond crossing bolted connections and to provide suggestions for improving their performance.

The ABAQUS environment is used to create and analyze the finite element model. A standard rail joint is modeled with shell elements in order to gain background knowledge on the performance of simple bolted connections in rail. The analysis of one crossing frog in a typical diamond crossing is then performed, again using shell elements. Static, linear elastic analyses were performed using amplified vertical wheel loads to approximate the dynamic effects. The crossing frog model is used to perform a parametric study that evaluates the relative influence of a variety of diamond crossing properties on the behavior of the diamond bolted connections. The results show that the longitudinal rail loads, vertical load position, and foundation stiffness significantly influence the stresses in bolts and connected elements.
Acknowledgments

I would like to thank my adviser, Professor Larry Fahnestock, for his guidance throughout this entire project as I believe it was a fascinating learning experience for both of us. I would also like to thank Riley Edwards for assisting the project along the way and providing some excellent contacts in the railroad industry.

I would also like to thank Chris Stoakes, Josh Steelman, and Jeff Meissner for helping me with any of my ABAQUS questions.

And of course, I must thank my parents and Emily for their constant support.
# Table of Contents

Chapter 1. Background and Literature Review

1.1 Introduction ........................................................................................................... 1

1.2 Literature Review ................................................................................................. 5
  1.2.1 Types of Diamond Crossings ......................................................................... 5
    1.2.1.1 Reversible Manganese Steel Inserts ................................................... 5
    1.2.1.2 Solid Manganese Castings ................................................................... 5
    1.2.1.3 Boltless Diamond Crossings ............................................................... 7
    1.2.1.4 Flange Bearing Frogs ......................................................................... 7
  1.2.2 Dynamic Vibration Response of Bolted Connections ...................................... 8
  1.2.3 Rail Height Mismatch for Standard Joints ....................................................... 9
  1.2.4 Patents Aiming to Improve Bolt Performance ................................................ 10
  1.2.5 Detailed Finite Element Studies of Standard Joints ...................................... 10
    1.2.5.1 Finite element analysis of bonded insulated rail joints ....................... 12
    1.2.5.2 Anatomy of joint bar failures ............................................................. 12
  1.3 Research Objectives ............................................................................................ 13

Chapter 2. Standard Joint Model

2.1 Overview ............................................................................................................... 14

2.2 Material Properties ............................................................................................ 14

2.3 Standard Joint Model Elements ......................................................................... 15
  2.3.1 Rails .............................................................................................................. 15
  2.3.2 Joint Bars ..................................................................................................... 16
  2.3.3 Bolts ............................................................................................................. 17

2.4 Assembly of Joint Elements ................................................................................ 18

2.5 Loads, Boundaries, and Support Conditions ..................................................... 20
  2.5.1 Boundary Conditions and Foundation Support ........................................... 20
  2.5.2 Loading and Load Steps ............................................................................. 21
    2.5.2.1 Bolt Preloading .................................................................................... 21
    2.5.2.2 Longitudinal Thermal Loading ......................................................... 22
    2.5.2.3 Vertical Wheel Loading .................................................................... 23
5.2.2 Effect of Loading ................................................................. 55
5.2.3 Load Case: Longitudinal Tension Only ........................................... 58
  5.2.3.1 Effect of Longitudinal and Lateral Restraint ............................... 58
  5.2.3.2 Effect of Tying Guard Rail and Diamond Casting ....................... 59
5.2.4 Load Case: Vertical Load Only ...................................................... 62
  5.2.4.1 General Behavior .................................................................. 63
  5.2.4.2 Position of Vertical Wheel Load ............................................. 65
  5.2.4.3 Effect of Base Plate Thickness ............................................... 66
  5.2.4.4 Effect of Foundation Stiffness ............................................... 67
Chapter 6. Summary, Conclusions, and Recommendations for Future Work .......... 70
  6.1 Conclusions .............................................................................. 70
  6.2 Recommendations for Future Research Investigating Diamond Crossings ....... 72
References .................................................................................. 74
Chapter 1. Background and Literature Review

1.1 Introduction

When two rail lines cross, they either meet at-grade or at a grade separation where a bridge carries one line over the other. However, grade separation is extremely expensive, time consuming to construct, and impractical for many situations. Most commonly the two rail lines will meet at-grade at a diamond crossing. Each diamond crossing contains four crossing frogs—one at each corner of the diamond, as shown in Figure 1. A typical frog in a diamond crossing is shown in Figure 2.

Figure 1. Example diamond crossing configuration (Tunningley et al. 1998).

Typical diamond crossing features include the frog castings, the rails that frame into the diamond, guard rails, and steel base plates. In most current diamond crossing configurations, the
castings and rails that compose the diamond are assembled with bolted connections. A guard rail is introduced as the open track approaches the diamond in order to reduce any lateral “hunting” of the wheels of the train. The space between the running rail and guard rail is called the flangeway. The intersecting rail also requires this flangeway, and the intersection of both of the flangeways occurs at the flangeway gap. When a train passes the crossing diamond, each wheel must “jump” the flangeway gap in each frog. However, as the wheel passes over the gap, its tendency to fall from the force of gravity coupled with the horizontal velocity of the train results in a significant impact on the frog.

Figure 2. Typical frog in a reversible manganese steel diamond crossing showing the intersection of two flangeways.

The repeated impact loading cycles create strong dynamic loading and amplification of the static wheel load on the diamond. Figure 3 shows plots of measured and theoretical amplification of standard vertical wheel loads. Plot (a) in Fig. 3 shows the measured amplification factor for conventional diamond crossings, while the plot below it provides theoretical amplified vertical wheel loads for three different crossing angles. A typical wheel
load for heavy haul freight lines is 39 kips (AREMA 2010), thus the bottom plot indicates an amplification factor of 120/39, which approximately equals 3 for 90-degree crossings (Shu and Davis 2008). At these high-angle crossings—where the intersecting rail lines are perpendicular—the amplification of load is highest because the wheel is entirely unsupported at the gap. For very low angle crossings, part of the wheel tread may be supported by the opposite side of the gap as the flange of the wheel is crossing the gap.

![DYNAMIC LOAD FACTOR](image1)

![DYNAMIC LOAD FACTOR](image2)

Figure 3. (a) Dynamic amplification of static wheel loads as a function of train speed (TTCI/AAR 2006). (b) Theoretical relationship of wheel load vs. train speed for crossing frogs (Shu & Davis 2008).
The amplified wheel loads and the dynamic loading cause problems for the rail car wheel assemblies, diamond crossing elements, and the ballast and foundation materials. Repeated load cycles may cause pumping and degradation of the ballast, resulting in settlement of the diamond with respect to the rail. This settlement only makes the load amplification worse over time. Additionally, the strong impact at the frog “point” causes significant wear and fracture of the manganese steel in the frog at the impact location. For the majority of crossing diamonds the individual castings are bolted together, and the repeated impact loads often result in loosening, fracture, and fatigue of the bolts and bolt-holes. Thus, crossing diamonds are much more expensive to maintain than open track (Armstrong 2008).

However, new diamond crossings are also expensive pieces of track work. Because they need to conform to the angle of each individual rail intersection, unique designs and drawings are required for nearly every different diamond produced by a manufacturer. This also means that new molds must be cast for each new design or for each different diamond angle. For each unique element of special track work created, the following process must be repeated for the manufacturer in question:

- A computer model of the casting is created
- The computer model is used to machine a full-size wooden model
- The wooden model is then used to create a mold composed of sand and an epoxy binder.
- Molten steel is then poured into the mold and the castings are allowed to cool.

This drives up the cost of diamonds and other special track work for the railroads. Therefore, it is clear that extending the service life of diamond crossings is important in driving down expenses.
1.2 Literature Review

1.2.1 Types of Diamond Crossings

There are many different diamond crossing designs used in practice. Discussions with industry professionals have indicated that the industry seems to cycle between complex designs with many parts and more simplified designs with fewer castings. Additionally, patents by Hein (2005, 2010), Tunningley et al. (1998), Grey (1997), and Collins (1982) have introduced a variety of different diamond crossing designs and innovations.

1.2.1.1 Reversible Manganese Steel Inserts

The reversible manganese steel casting is used as part of a more complicated, modular diamond crossing. High-manganese steel and explosive depth hardening is used in these castings in an attempt to delay the material deterioration shown in Figure 4, which shows a typical diamond crossing frog utilizing the manganese casting inserts (Gregory and Palmer 1994). At each frog in a diamond, three of the frog “corners” will receive wheel impacts and one corner will receive no impacts. Additionally, the corner opposite the one that receives no impacts will be impacted by wheels traveling along both of the intersecting rail lines. The advantage of this modular, reversible design is the ability to remove an inner casting and rotate or switch it with the casting at the opposite corner of the diamond. Thus, the point that received no impacts can then be made the point that receives the double impacts, theoretically extending the service life of the castings.

1.2.1.2 Solid Manganese Castings

Alternatively, simpler designs use only four solid manganese castings—one for each corner of the diamond. A plan view of a diamond crossing utilizing this type of casting is shown
in Figure 5. A close-up of one corner of this type of diamond is also shown in Figure 13. There are fewer moving parts in this kind of diamond, and thus fewer individual elements that must be kept bolted tightly together. The idea in this case is that it is easier to maintain alignment and joint tightness by using the solid manganese castings. However, maintaining alignment is not an easy task. Additionally, these castings are not reversible and thus, when they wear out, the diamond must be replaced. The shaded regions in Figure 5 identify the locations of wheel impact, showing that there is in fact one point that is not impacted.

Figure 4. Reversible manganese steel frog casting in a diamond, showing fracture of material at point of impact (J. Riley Edwards personal communication 2010).

Figure 5. Plan view of diamond using solid manganese steel casting design. Shaded regions indicate depth-hardening of manganese steel at impact locations (J. Riley Edwards personal communication 2010).
1.2.1.3 Boltless Diamond Crossings

In the previous two cases, the different diamond elements are bolted together. The lap beam design introduced by Tunningley et al. (1998) attempts to dispatch of the bolted connections in the diamond crossing completely. This design considers the diamond as composed of four “crossing beams.” Thus, the train runs along the rail as it approaches the diamond, then transitions onto the crossing beams (which contain a flangeway) as it moves through the diamond. Each crossing beam has cutouts to make room for the intersecting crossing beams, and the beams are then pinned together (Tunningley et al. 1998). Though the simplicity of this design is attractive, industry professionals have indicated that it is not commonly used because cracks in the crossing beams would necessitate expensive replacement of the diamond. Thus, although this design initially received attention when it was introduced, failure of the lap beams caused the industry to revert to the simpler designs and favor a more modular approach.

A variation on the lap beam design was developed by Hein (2005). This design features a number of beam castings which are interchangeable with each other, and are fastened to baseplate with clips and wedges that resist horizontal and vertical movement. This is a fairly geometrically simple design introduced in an attempt to devise a standardized crossing diamond design that could be used for a wide range of crossings and angles (Hein 2005). However, this design contains large beam castings that may be susceptible to fatigue cracking.

1.2.1.4 Flange Bearing Frogs

Recently, flange-bearing frogs were introduced and have been used to greatly reduce the impact loadings by essentially eliminating the gap that the wheels have to “jump.” A flange bearing frog design was introduced in a patent by Robert Willow (1996). The flangeway gap is still in place, but the flangeway is raised on the diamond approaches so that the wheels ride on
their flanges as they pass through the frog. These flange-bearing frogs are expected to have lower life cycle costs because there is no sudden impact, or the impact is significantly lessened, when the wheels pass over the frog (Gregory and Palmer 1994). However, diamond crossings with flange-bearing frogs have a higher initial cost than conventional diamonds. Additionally, some speed restrictions may be imposed on trains passing over full flange-bearing diamonds. One-way low speed (OWLS) diamonds are another relatively new innovation. In this design, one of the rail lines is continuous without any gap that must be jumped. Its flangeway must be jumped by the wheel flange of the intersecting line, however. Thus, the intersecting line is restricted to very low speeds when passing over the diamond because it must “jump” a flangeway and the impact occurs on the wheel flange. Therefore, OWLS diamonds are only suitable for crossings in which one of the intersecting lines carries much less load and can cross the diamond at a low speed (Davis et al. 2006).

1.2.2 Dynamic Vibration Response of Bolted Connections

Because of the dynamic loading and vibration caused by the repeated wheel impacts on the frogs, literature examining the behavior of bolted connections under dynamic loads was explored. Ramey and Jenkins (1995) observed the performance of threaded connections when subjected to dynamic vibrations. They performed vibration experiments on 96 different test cases in which they varied parameters such as the bolt diameter, bolt lubrication, initial bolt preload, hole tolerance, loading configuration, and nut locking device. They did not observe much bolt loosening in their tests, but were able to make some conclusions. They suggested that it is desirable to preload bolts to loads that are as high as possible (close to the bolt yield strength), and it is important to maintain a high level of the initial preload on the bolt. In order to maintain preload, they suggest using toothed washers and including some method of vibration
damping in the system. They also suggest that transverse bolt loads and impact loads contribute heavily to bolt loosening (Ramey and Jenkins 1995). Unfortunately, bolts in diamond crossings likely incur significant transverse loading and unloading as trains pass over the diamond. Thus, these loading conditions are unavoidable.

1.2.3 Rail Height Mismatch for Standard Joints

Mayville and Stringfellow (1995) studied the performance of bolted standard joints subjected to wheel load impacts arising from joint looseness and mismatch of the rail heights at the joint. They sought to determine impact of these parameters on the propensity for crack propagation at a bolt hole. Their investigation utilized a finite element model composed of beam elements to model the rails and joint bar. They concluded that loosening of the joint increased the dynamic load on the joint and increased the likelihood of fracture at the bolt hole. These results reflected the observations of railroad practitioners. However, their finite element analysis also found that increased joint looseness resulted in a corresponding decrease in the stress intensity factor at the bolt hole due to static load only. Additionally, Mayville and Stringfellow found that increasing rail height mismatch also increased the shear force at the bolt hole. This impact due to a rail height mismatch could be analogized to the impact due to the passage over the flangeway gap in a diamond crossing. It can be hypothesized that these impacts may cause loosening of the bolts in the crossing, which in turn may increase the potential for cracks to grow around the bolt holes. Additionally, the large change in modulus experienced by train cars when traveling from open track to a crossing diamond may introduce a dynamic loading situation that is very unfavorable for the diamond elements.

Cai et al. (2007) also examined mismatch of the ends of the rail at a standard joint. However, their investigation focused on contact stresses between the wheel and rail using a
three-dimensional finite element model composed of solid elements. As expected, they found that increasing train speeds and greater height mismatch resulted in larger contact stresses between the wheel and rail.

**1.2.4 Patents Aiming to Improve Bolt Performance**

Patents by Grey (1997) and Atkinson (1998) show that the industry does place importance on maintaining snugness and perpendicularity of fit in crossing frog bolted connections. Grey’s bolted connection assembly uses a square lock nut that deforms the bolt shank threads once it is screwed onto the bolt in an attempt to prevent bolt loosening and retain alignment (1997). The bolt-nut assembly introduced by Atkinson for special track work applications utilizes a design in which the inner surfaces of the bolt head and nut are radiused. This is combined with a beveled washer to provide a snug fit. Even if alignment of the track elements is not perfect, the connection will maintain a tight fit because of the circular shape of the fit between bolt and washer (Atkinson 1998).

**1.2.5 Detailed Finite Element Studies of Standard Joints**

In a review of prior work and research on crossing diamonds, no prior finite element modeling studies were found examining the bolted connections in special track work. Studies by Xiao et al. (2011) and Guo et al. (2010) investigated the contact stresses between the wheel and crossing frog as a train passes over a crossing diamond. Shu et al. (2011) investigated the performance of rails with gaps cut into the rail head at their TTCI testing site. The cutout gaps were used to model the jump that occurs at the flangeway gap. Using strain gages and accelerometers attached to the passing trains, they found that including a joint at the gap actually reduced dynamic loading. Therefore, it was hypothesized that perhaps a jointed frog could mitigate the dynamic effect of the wheel load by providing damping to the frog.
However, there is still a paucity of published research examining the behavior of crossing frog or crossing diamond bolted connections with computer models or field measurements. Additionally, there are very few publications that include quantified measurements of crossing diamond behavior in the field. From speaking with experts in the field, much of the knowledge in the railroad industry regarding crossing diamond behavior seems to be based on experience and relayed by word of mouth. Therefore, there is a need for further finite element analysis of crossing diamonds in order to quantify their behavior.

A series of studies, which used finite element models, studied standard bolted rail joints. Igwemezie and Nguyen (2009a) investigated the performance of standard bolted joints and concluded that many of these joints in the field are deficient for modern wheel loads. They proposed modified joint bar details to mitigate the deficiencies in the field and in new designs (Igwemezie and Nguyen 2009b, 2010). There have also been several investigations into the behavior of bonded insulated rail joints. Himebaugh et al. (2008) investigated the state of stress in the insulated joint epoxy layer under vertical and horizontal loading. Kerr and Cox (1999) created an analytical model to determine the deflection of an insulated joint. Plaut et al. (2007) performed two different analyses on a boltless, tapered insulated joint, in which the ends of the rail at the joint are cut at an angle. They first performed an analysis using the Rayleigh-Ritz method. This analysis modeled the joint bars and rails as beam elements, connecting the rails and joint bars with springs and using an elastic foundation to connect the rail to the ground. The second part of their study used a finite element model composed of solid elements to examine the shear stress at the interface between the rails and bars at the joint. Peltier et al. (2007) focused their investigations on characterizing the debonding of the epoxy layer between the joint bar and
They used a finite element model and experimental tests to measure the shear strain in bonded insulated joints and examine the debonding of the epoxy layer.

After performing this overview of prior research on bolted connections in rail, it was concluded that the “Finite element analysis of bonded insulated rail joints” article by Himebaugh, Plaut, and Dillard (2008) and the “Anatomy of joint bar failures” articles by Igwemezie and Nguyen (2009a, 2009b, 2010) provided the closest models for the bolted connections examined in this paper. These studies are briefly discussed in more detail in the following sections.

1.2.5.1 Finite element analysis of bonded insulated rail joints

Himebaugh (2006) performed a finite element analysis on bonded, insulated joints. These joints are often used around diamond crossings in order to create signal blocks for detection of train locations and breaks in rails (Himebaugh et al. 2008). In this joint, the epoxy connecting the joint bars and rail is assumed to carry the entire wheel load and thermal load (from contraction of the rails). A sophisticated three-dimensional model was created using solid finite elements to determine the stresses in the epoxy layer. Though this type of joint does not rely on bolts to carry load, the global behavior of the joint is similar to that of standard bolted joint.

1.2.5.2 Anatomy of joint bar failures

Igwemezie and Nguyen (2009a) performed finite element analyses to examine the behavior of standard, bolted rail butt joints. They also performed finite element investigations into temporary joint-strengthening solutions and new joint designs (2009b, 2010). The goal of their research was to devise a new joint design that would improve the performance of bolted rail-rail joints in the field under ever-increasing axle loads. Igwemezie and Nguyen also used three-dimensional solid elements to create realistic models of the joint elements. Their analysis
considered both winter and summer conditions, particularly because a larger portion of joint failures occur during the winter (Igwemezie and Nguyen 2009a). In the winter, the foundation provides added stiffness but there is a large tensile, longitudinal load imposed on the joint due to the tendency of the rails to shrink under very cold temperatures. Their first study concluded that the standard joint used in practice for many years is structurally deficient for the current vertical loads. Additionally, they noted that thermal stresses in winter months can result in very large tensile loads on the joint that significantly contribute to joint failures.

1.3 Research Objectives

The focus of this research project is an investigation into the general behavior of bolted diamond crossing connections using finite element analysis. The influence of a number of parameters on bolted connection behavior under static loading is characterized. In particular, the results from Igwemezie and Nguyen (2009a) indicate that the presence of thermal, tensile loads entering diamond crossings could increase the propensity for failure of the bolted connections. Thus, the role of both vertical and horizontal loads is investigated. The analysis results are combined with insight from the literature review to provide pragmatic suggestions for improving bolted connection performance in special track work and diamond crossings.
Chapter 2. Standard Joint Model

2.1 Overview

Due to the complex geometry of diamond crossings, it was deemed impractical to create finite element models composed of three-dimensional solid elements that could be analyzed efficiently and used to study parametric variations. Instead, shell element models where chosen since they offer the ability to create relatively complex and realistic three-dimensional structures that can be analyzed much more quickly and still provide reasonable results for studying the effect of critical parameters. The following sections describe the methodology behind the creation of rail elements for use in a standard joint model utilizing shell elements. This was performed in order to investigate the effectiveness of shells in modeling rail elements and to compare the results to the investigations by Igwemezie and Nguyen (2009a) and Himebaugh (2006).

2.2 Material Properties

The ABAQUS/CAE environment was used to create and analyze all models under static conditions. Since linear material behavior was considered, only the elastic modulus, \( E \), and Poisson’s ratio, \( \nu \), were required. For rail steel and manganese steel, \( E = 30,000 \text{ ksi} \) and \( \nu = 0.3 \) were used. Because the joint geometry contains contact between joint bars, bolts, and rails, a friction coefficient for contact definitions was also needed. For steel-on-steel contact, a friction coefficient \( \mu = 0.35 \) was used. The penalty method was used for all contact definitions.
2.3 Standard Joint Model Elements

The following sections describe the basis and strategy for modeling the individual parts and assembling the rail joint. Following that is an examination of the results and a comparison to the previous research.

2.3.1 Rails

The rails were treated in the same way that steel I-shaped sections are commonly modeled using shell elements. In the case of typical steel I-shaped sections, two-dimensional shells and plates are appropriate because the flange and web thicknesses are usually fairly small compared to their width dimensions. In the case of rails, that is not the case, but shell elements with thick formulations are used to approximate the properties of the rails. Therefore, exact stresses on a local scale are not considered to have high fidelity in regions of the model where the thickness is large with respect to the width (the head and foot). Instead, the focus was placed on the global behavior of the joint and some detailed observations were made at the bolted connections where the rail sections are thinner (the web).

AREMA provides a suggested section for the 136 RE rail section used in this model (2010). The complex geometry of a piece of the 136-lb rail was simplified into three sections representing the head, web, and foot. Because this investigation focused on the response of the rail to vertical load as well as axial load, it was most important to match the area and moment of inertia of the real section. The approximate section is shown with the suggested section provided by AREMA for 136-lb rail in Figure 6. The web height is 4.3,” the web thickness is 0.6,” the head thickness is 1.8,” and the foot thickness is 0.9.” Thus the total height of the section is 7.0.” The area of the section and moment of inertia about its horizontal neutral axis closely reflect the properties of the real section, shown in Figure 6 and Table 1.
Figure 6. (a) AREMA (2010) suggested 136RE rail section. (b) ABAQUS model.

Table 1. Comparison of model and AREMA (2010) rail section properties.

<table>
<thead>
<tr>
<th></th>
<th>AREMA</th>
<th>Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moment of Inertia, I</td>
<td>94.20 in^4</td>
<td>92.09 in^4</td>
</tr>
<tr>
<td>Area, A</td>
<td>13.33 in^2</td>
<td>13.38 in^2</td>
</tr>
<tr>
<td>centroid (from foot)</td>
<td>3.34 in</td>
<td>3.23 in</td>
</tr>
</tbody>
</table>

2.3.2 Joint Bars

The shape of the joint bar largely depends on its intended usage. The standard joint bars studied in the “Anatomy of joint bar failures” research have a “dog bone” or “I” shape and only contact the “corners” of the rail sections (Igwemezie and Nguyen 2009). On the other hand, the joint bars used for bonded insulated joints must be flush with the entire height of the rail web. This must be the case because the epoxy that bonds the joint bars to the rail is the chief carrier of
load between the two elements. The shape of the shell-rail section forces the joint bars to be 4.3” tall. In order to retain the relative dimensions of the joint section, the joint bar section was chosen to be rectangular in shape, with a thickness of 1.4.” Additionally, the joint bar was fit snug with the rail web along its height, with the bar and rail in contact. The top and bottom of the joint bars also fit snug into the space between the bottom of the rail head and the top of the rail foot. The joint bars are 36” (3’) long with six bolt holes (1 1/8” diameter). The bolt holes are spaced 6” on center, with the end bolts 3” on center from the edge of the joint bar.

2.3.3 Bolts

1”-diameter bolts were chosen for this joint. The shanks of the bolts were modeled using beams with a circular section. The bolt head and nut were modeled using shell elements, enabling contact between the head and nut with the joint bars. The diameter of the nut/head is 1½ inches while the bolt shank has a diameter of 1 in. The bolt nut and head are each 1” thick. Because the joint consists of a 0.6”-thick rail web and two 1.4”-thick joint bars, the bolt shanks are 3.4”-long from the inner surface of the nut to the inner surface of the head. Any bolt material that extends beyond the nut was not considered or modeled.

The bolt shanks were meshed with linear, shear-flexible beam elements. These elements are referred to as B31 linear beams in ABAQUS. The nut and head are meshed with triangular shell elements (S3). Additionally, a material with elastic modulus of $1 \times 10^{10}$ psi was used for the inner 1”-diameter of the bolt head and the nut. This is done since in reality the shank of the bolt provides stiffness to the inner portion of the head, but this restraint is not modeled when the shank beam element only shares a single node with the shell elements representing the nut and head.
2.4 Assembly of Joint Elements

The Assembly and Interaction stages in ABAQUS/CAE were used to define the initial locations of each element of the structure and the connection between these elements. A section view of the assembled joint is shown in Figure 7. The most significant issue considered in these stages was the transfer of force from the rail and joint bars to the bolts. Since solid finite elements were not used in the analysis, explicit contact definitions could not be used to model the bearing of the bolt on the bolt hole. Instead, intermediate elements were used to represent the interaction.

![Figure 7. Section view of the joint assembly rendering in ABAQUS.](image)

Axial (truss) elements were used to connect the degrees-of-freedom at a point on the bolt shank to degrees-of-freedom at a point around the bolt hole. The axial trusses were positioned radially around the bolt. Initially, 4 radial trusses were used at each bolt hole to transfer vertical and horizontal force between the bolts and rails/joint bars. Thus, the trusses were spaced 90 degrees radially apart. Because the goal was to approximate the hard contact between the bolt and bolt hole, the truss elements were given a section with unit area and a very high elastic
modulus on the order of $1 \times 10^8$ ksi. Because this connection simulates bearing of the bolt on the hole, the truss material was not allowed to provide any resistance in tension. Therefore, when the bolt tries to move in one direction, stresses do not develop on both sides of the hole but only at the location where bearing would occur.

However, limiting the number of radial truss elements at each bolt hole also limits the load to flowing in user-defined paths from bolt to joint bar. Ideally it would be advantageous to have as many radial truss elements as possible at each bolt hole. By radially adding more truss elements, the load is transferred between bolt and hole at locations where bolt bearing would naturally occur. This provides a more accurate “map” of the load path between the bolt shanks and the bolt holes in the rails and joint bar, but it is not an efficient modeling approach, particularly for joints with many bolts. Since the special track work models used in this research were developed to evaluate more global system behavior and to compare results while adjusting a variety of parameters, reduced accuracy for local stress and strain data due to coarse element density was acceptable.

Load transfer largely occurs at the edges of the bolt hole section, as shown in the finite element analyses performed by Igwemezie and Nguyen (2009a). Since shell finite elements are defined on a single plane and connections must occur at a point on the reference plane, a stiff beam element was used to model the width of the rail web or joint bars at the interface. This beam element is perpendicular to the plane of the shell and is the same length as the thickness of the shell element to which it is connected. The axial truss elements are then connected from the bolt shank to this stiff beam element. For every point around the bolt hole that there was just one axial element before, now there are three axial truss elements in parallel connected to a stiff beam. This pattern is repeated for as many locations around the bolt as desired. In the case of
the standard joint model, the truss elements are spaced 45 degrees apart radially around the bolt hole. This method yields more realistic results, transferring a greater amount of shear force to the bolt at the interface of joint bar and rail. However, this again requires many more elements, more connectors, and makes the model less efficient.

Contact definitions were used in ABAQUS to model interaction between the bolt nut, joint bars, and rail. Therefore, in the model, frictional resistance is provided by the flat surface-to-surface contact between the joint bar and rail web. Additionally, in order to model the bearing between the rail and joint bars, the nodes along top and bottom edges of the joint bars were defined in contact with the underside of the rail head and the top side of the rail foot. Thus, there will be bending about the axis that is perpendicular to the joint bar face. Hard contact was defined using the penalty method in ABAQUS, and a coefficient of friction of 0.35 was used to model steel-steel contact.

2.5 Loads, Boundaries, and Support Conditions

2.5.1 Boundary Conditions and Foundation Support

Boundary conditions were applied at the far end (as opposed to the joint end) of each rail. All degrees-of-freedom were restrained except for the degree-of-freedom corresponding to the longitudinal axis of the rail. This was left unrestrained so that an axial force could be applied to the rail, as discussed in 2.5.2.2.

To model the support provided by the ballast and ties, the rails are supported by an elastic foundation at the tie locations. Kerr and Cox (1999) analyzed a standard joint using beam elements supported on a continuous elastic foundation with stiffness $k = 3,000$ psi. Igwemezie and Nguyen (2009a) used elastic foundations of stiffness $k = 1,500$ psi for summer and 4,500 psi for winter. In a 2-D beam on elastic foundation model, the rail is supported continuously by an
elastic foundation of stiffness $k$ (with units of force per length per unit length of the beam). In the three-dimensional model, this value is used to find the foundation stiffness per unit area under the rails. For 10”-wide ties spaced 20” on center, the effective elastic foundation stiffness, $k_{\text{eff}}$ is:

$$k_{\text{eff}} = \frac{k \times s}{A_{\text{tie}}}$$  \hspace{1cm} (1)

where:

$s = $ tie spacing

$A_{\text{tie}} = $ Area of rail supported by each tie = rail foot width $\times$ tie width

A tie spacing of 20” was chosen and both 9”-wide and 11”-wide ties were used depending on the support condition at the joint. Thus, Eqn. 1 yields results of $k_{\text{eff}} = 111$ lb/in/in² for the 9” ties and 909 lb/in/in² for the 11” ties.

2.5.2 Loading and Load Steps

The following sections describe the loads applied to the structure. The loading is carried out in three separate ABAQUS steps. First, the bolts are pre-tensioned. Next, the longitudinal tension is applied to the joint. Thus, performance of the joint under the static thermal tension can be observed. This load step is disabled in the cases where performance under relaxed, summer conditions is investigated. The final step is the introduction of the vertical wheel load at the joint.

2.5.2.1 Bolt Preloading

In order to tension the bolts, a temperature field is applied to the bolt shank beam elements. The amount of temperature change needed to achieve a certain pre-load can be
calculated using simple solid mechanics formulas for the amount of force in a fixed axial 
member subjected to a change in temperature from its neutral state.

The “fixed-end force” in an axial member due to a temperature differential is given by 
the following formula:

\[ F = AE\alpha\Delta T \]  
(2)

In Eqn. 2, \( F \) represents the bolt pre-tension or preload, \( A \) is the cross-sectional area, \( E \) is the 
elastic modulus, \( \alpha \) is the coefficient of thermal expansion, and \( \Delta T \) is the temperature change 
applied to the member. For the 1”-diameter steel bolts, the following properties were used:

\[ A = \frac{\pi}{4} (1")^2 = 0.785 \text{ in}^2 \]
\[ E = 30,000 \text{ ksi} \]
\[ \alpha = 6.5 \times 10^{-6} \text{ in/in/°F} \]
\[ F = 30 \text{ kips} \]

The desired pretension (∆\( T \)) used is 30 kips, which falls on the high end of the AREMA (2010) 
specification of 20,000 to 30,000 lbs per track bolt. However, some of this load will be lost due 
to elastic deformations of the bolt and rail components. Eqn. 1 can be re-arranged to solve for 
the temperature field that needs to be applied to each bolt.

\[ \Delta T = \frac{F}{AE\alpha} = \frac{30 \text{ kips}}{(0.785 \text{ in}^2)(30,000 \text{ ksi})(6.5 \times 10^{-6} \text{ °F}^{-1})} = 196 \text{ °F} \]  
(3)

2.5.2.2 Longitudinal Thermal Loading

After bolt tensioning is completed, the longitudinal load is applied to the ends of the rail 
for the cases in which thermal loading is considered. This axial force represents the thermal 
stress that develops during expansion and contraction of the continuous rail segment. During the 
summer, the rail is considered to exist in its relaxed state. However, during the cold winter
months, the rails will contract, resulting in very significant tensile forces on the joint. The load was applied as a point load at the rail centroid and as a distributed load over the height of the web. In both of these cases, the response near the joint was the same because the stresses redistribute evenly across the section away from the boundary where the load is applied. The two finite element studies used for comparison to the current research used slightly different longitudinal rail loads. Igwemezie and Nguyen (2009a) considered a 200 kip tension load applied on the joint, while Himebaugh (2006) used a tensile longitudinal load of 300 kips.

2.5.2.3 Vertical Wheel Loading

The two prior studies that were used as benchmarks used different wheel loads. The present research matched the appropriate longitudinal and vertical wheel loads of the prior research investigations in order to directly compare results. Igwemezie and Nguyen (2009a) studied a standard joint subjected to a 44 kip vertical load, whereas Himebaugh (2006) used a wheel load of 32.5 kips. A vertical wheel load was applied as a concentrated force at the top of one of the rails at the joint. An ABAQUS equation constraint was then used to tie the vertical degree-of-freedom of the same point on the adjacent rail, thereby distributing the concentrated force evenly to both rails.

2.5.3 Meshed Assembly

Figure 8 shows an image of the final finite element mesh of the structure near the joint. Multiple meshes were used and a mesh refinement study is included in section 3.2.
Chapter 3. Standard Joint Model Results

The following sections describe the behavior of the standard joint finite element model created for this research. Global deformations are examined and compared to the results of the reference studies. Additionally, some conclusions will be drawn from the behavior of the joint model and applied to the subsequent modeling of the diamond crossing elements. The behavior of the model with respect to the assumptions and simplifications will be observed in order to understand any limitations of shells in modeling railway elements.

First, however, the rail developed in 2.3.1 was subjected to a vertical wheel loading to compare the response of a single, continuous piece of rail modeled with shells with the theoretical beam on elastic foundation solution developed by Hetényi (1979). The boundary conditions were then changed to a fixed-end beam to compare vertical deflection to predicted deflection using structural analysis. This process is performed to show that the shell element rail section can be used to accurately model global bending behavior.

3.1 Continuous Rail Subjected to Vertical Wheel Load

The response of a continuously supported segment of the rail under vertical load was examined to compare to the theoretical solution for a beam on elastic foundation developed by Hetényi. Various lengths of rail were examined, and the deflections for both a 480”-long and 800”-long segments of rail are provided. First, however, the beam on elastic foundation theory is described.
3.1.1 Hetényi Beam on Elastic Foundation

Hetényi (1979) developed an exact solution for a beam of unlimited length supported continuously by an elastic foundation, subject to a point load. To do this, a solution was developed for the differential equation describing the deflection of a beam:

\[ EI \frac{d^4y}{dx^4} = -ky \]  \hspace{1cm} (4)

In Eqn. 4, \( y \) is the vertical deflection of the beam and \( k \) is the “modulus of the foundation” in units of force/length/length. The general solution to this equation is

\[ y = e^{\lambda x} (C_1 \cos \lambda x + C_2 \sin \lambda x) + e^{-\lambda x} (C_3 \cos \lambda x + C_4 \sin \lambda x) \] \hspace{1cm} (5)

where

\[ \lambda = \left( \frac{k}{4EI} \right)^{\frac{1}{4}} \] \hspace{1cm} (6)

Now consider the case where there is a point load applied at \( x = 0 \). As \( x \to \infty \), \( y \) must approach 0. However, as \( x \to \infty \), \( e^{\lambda x} \to \infty \) (since \( \lambda \) is always greater than 0). Therefore, \( C_3 \) and \( C_4 \) must be equal to 0. Additionally, at \( x = 0 \), \( \frac{dy}{dx} = 0 \). Carrying out the derivative and substituting 0 for \( x \) shows that \( C_3 = C_4 = C \). Finally, to solve for \( C \), it is noted that the total force carried by the elastic foundation must be equal to the applied load \( P \) in order to maintain equilibrium:

\[ \int_0^\infty ky \, dx = \frac{P}{2} \] \hspace{1cm} (7)

\[ 2kC \int_0^\infty e^{-\lambda x} (\cos \lambda x + \sin \lambda x) \, dx = P = 2kC \left( \frac{1}{\lambda} \right) \] \hspace{1cm} (8)
Therefore, \( C = P\lambda / 2k \) and the deflection of the beam (for \( x > 0 \)), is given by the equation

\[
y = \left( \frac{P\lambda}{2k} \right) e^{-\lambda x} (\cos \lambda x + \sin \lambda x)
\]

(9)

The condition considered is \( P = 44,000 \) lbs and \( k = 3,000 \) psi, giving a theoretical deflection of \( y = 0.167'' \) (downward deflection is positive) at \( x = 0 \).

### 3.1.2 Shell Element Model

The segment of rail without a joint was continuously supported at the 6"-wide foot by an elastic foundation for the entire length of the rail. Thus, the elastic foundation has a value of \( 3000 \) psi / 6" = 500 lb/in/in\(^2\). The deflection of the bottom of the rail is compared to the beam on elastic foundation solution, and the results are shown in Figure 9.

![Right-Side Deflection of Single Piece of Rail](image)

Figure 9. Plot of right-hand side of a continuous rail supported by elastic foundation, showing comparison between theoretical infinite beam solution and the finite element model solution.
The figure on the previous page is a plot of the vertical deflection of one half of a rail segment subjected to the 44 kip point load, applied at \( x = 0 \). The “Hetényi Solution” curve corresponds to the theoretical solution presented in the previous section; this curve was drawn using Eqn. 9. The two curves representing model results lie almost exactly on top of one another, as the model has essentially converged when the rail is 480” long: the 800” case is almost exactly the same. That is, the rail is long enough that the solution has essentially converged to the model case of an unlimited length rail. However, there are some differences between the theoretical solution and the measured response of the finite element model. Most notably, the model over-estimates the maximum deflection near \( x = 0 \) (directly beneath the point load). Therefore, a study without the elastic foundation was conducted to determine whether the rail was behaving properly in bending. At the ends of the rail, the boundary conditions were fixed in all degrees-of-freedom. The elastic foundation was removed and the rail subjected to the same 44 kip load as before. The analysis was performed and the deflection at \( x = 0 \) was measured to be equal 42.67”. The exact solution for the deflection of a fixed-fixed beam directly below a load at the centerline is given by the formula

\[
y = \frac{PL^3}{192EI} = \frac{44 \text{ kip} \times (800”)^3}{192 \times (30,000 \text{ ksi})(92.09 \text{ in}^4)} = 42.47”
\]  

(10)

The model solution very closely matches the exact deflection. Therefore, it can be concluded that the rail section behaves properly in bending.

### 3.2 Mesh Refinement of the Standard Joint Model

Now, the response of the standard joint model under vertical loading is considered. The following section considers the effects of mesh refinement on vertical deflection of the standard joint. The plot in Figure 10 shows the vertical deflection of one rail (\( x = 0 \) again corresponding
to the centerline of the model) when using three different meshes. “Fine” refers to the mesh with the most elements, while “Coarse” refers to the least refined mesh. Table 2 details the number of elements constituting the rail section for each mesh. The “BOEF” curve corresponds to the Hetényi (1979) beam on elastic foundation model for a piece of straight rail without a joint, the formula for which is found in Eqn. 9. The joint should deflect significantly more than the infinite beam under loading and the model results reflect this. Additionally, the mesh refinement results in very little improvement in the accuracy of the vertical deflection. Although the global deflection changed very little during mesh refinement, the finest mesh did result in some small improvements near the application of load.

Table 2. Rail section mesh details.

<table>
<thead>
<tr>
<th>Rail Section Feature</th>
<th>Coarse</th>
<th>Medium</th>
<th>Fine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Web Height</td>
<td>4</td>
<td>4</td>
<td>7</td>
</tr>
<tr>
<td>Foot Width</td>
<td>4</td>
<td>8</td>
<td>12</td>
</tr>
<tr>
<td>Head Width</td>
<td>2</td>
<td>4</td>
<td>6</td>
</tr>
</tbody>
</table>

Table 3 and the plot in Figure 10 show that there is essentially a negligible improvement in the measurement of deflections when moving from the coarsest mesh to the finest mesh. However, in order to present a better picture of the stress contours in the bolted connections at railroad joints, the fine mesh is needed.

Table 3. Elements in each mesh and percent difference in deflection of successive meshes.

<table>
<thead>
<tr>
<th>Mesh</th>
<th>Number of Elements</th>
<th>Max. Vertical Deflection (in.)</th>
<th>Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Coarse</td>
<td>5324</td>
<td>0.3617</td>
<td>-</td>
</tr>
<tr>
<td>Medium</td>
<td>10264</td>
<td>0.3594</td>
<td>0.63995548</td>
</tr>
<tr>
<td>Fine</td>
<td>23286</td>
<td>0.3588</td>
<td>0.16722408</td>
</tr>
</tbody>
</table>
3.3 Comparison of Results to Existing Research

The following section focuses on the results of the model compared to prior finite element analyses published by Igwemezie and Nguyen (2009a) and Himebaugh et al. (2008). The different loading and support conditions used in each of those studies were replicated here and applied to the standard joint model. The results are observed and their meaning discussed.

3.3.1 Standard Joint Analysis Cases

In the study “Anatomy of joint bar failures I,” Igwemezie and Nguyen (2009a) investigated a standard joint configuration connecting two sections of open track. A wheel load of 44,000 lbs and a longitudinal thermal-induced load of 200,000 lbs were used. Additionally, foundation stiffnesses of 1,500 psi and 4,500 psi were used for summer conditions and winter conditions, respectively. The summer conditions consider the rails to be in a relaxed state where
no longitudinal load is applied to the joint. The winter conditions consider a stiffer foundation support, but also the presence of the tensile longitudinal load due to contraction of the rails. A tie spacing of 24” was used but the tie width was not provided. The total length of the structure was 480”, in which two 240” pieces of rail were connected at the standard joint. Additionally, Igwemezie and Nguyen (2009a) only considered the case where the ends of the rail are suspended by the rail ties at the connection. That is, there is no tie directly beneath the ends of each rail.

Himebaugh et al. (2008) investigated insulated rail joints in their study titled “Finite element analysis of bonded insulated rail joints.” In these insulated joints, the epoxy between the joint bars and rail carry the entire load. Still, their general behavior is similar and is a useful point of comparison for calibration of the model in the present research. In Himebaugh et al (2008), a longitudinal thermal load of 300,000 lbs and a vertical wheel load of 32,500 lbs were used. Ties of width 11 inches and 9 inches were investigated, with the joint “supported” by a tie in all cases. Two 300”-long pieces of rail were joined, bringing the total structure length to 600” in this case.

Table 4. Loading cases considered for standard joint finite element model.

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Joint Support</th>
<th>Vertical Load</th>
<th>Longitudinal Load</th>
<th>Foundation Stiffness</th>
<th>Tie width</th>
<th>Comparison Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Suspended</td>
<td>44 kips</td>
<td>0 kips</td>
<td>1500 psi</td>
<td>9 in.</td>
<td>Igwemezie</td>
</tr>
<tr>
<td>2</td>
<td>Suspended</td>
<td>44 kips</td>
<td>200 kips</td>
<td>4500 psi</td>
<td>9 in.</td>
<td>Igwemezie</td>
</tr>
<tr>
<td>3</td>
<td>Supported</td>
<td>32.5 kips</td>
<td>0 kips</td>
<td>3000 psi</td>
<td>11 in.</td>
<td>Himebaugh</td>
</tr>
<tr>
<td>4</td>
<td>Supported</td>
<td>32.5 kips</td>
<td>300 kips</td>
<td>3000 psi</td>
<td>11 in.</td>
<td>Himebaugh</td>
</tr>
</tbody>
</table>

Table 4 summarizes the load and support conditions imposed on the standard joint model. The first column, labeled “Joint Support,” describes the tie condition directly beneath the joint between the two rails. “Suspended” denotes the case where there is no tie directly beneath the joint and the two rails are slightly “cantilevered” from the ties. Thus, in this case, there was no
elastic foundation supporting the rails at the joint. All of the models analyzed by Igwemezie and Nguyen (2009a) were conducted with this support condition, therefore this was imposed on the current model in order to compare the results to that study. Himebaugh (2006) studied the case where the joint was “supported,” in which case there is a tie directly beneath the joint and therefore the elastic foundation is applied directly at the joint as well.

The “Vertical Load” column denotes the vertical wheel load, which was applied directly over the joint in all cases. “Longitudinal Load” is the tensile, axial load applied to the rail section to simulate thermal contraction of the rail as described in section 2.5.2.2. “Foundation Stiffness” refers to the equivalent $k$ value for a beam on elastic foundation before it has been converted to the $k_{eff}$ applied at the tie locations. “Tie width” describes the width of the railroad cross-ties simulated by the elastic foundation. The first two load cases compare the model results to Igwemezie and Nguyen (2009a), while the third and fourth tests use the load and support conditions of Himebaugh et al. (2008).

The ABAQUS model created for the present research joined two 240” pieces of rail, with a 0.875” gap separating the rails at the joint. The tie spacing was 20” for all cases, with 11” and 9” wide ties being considered. Therefore, the model does not exactly match the conditions for both of the prior research articles. The foundation or tie supports most closely match those used in Himebaugh (2006), but the rails in the present model are shorter. This has little effect on the results, however, as previously discussed and portrayed in Figure 9.

### 3.3.2 Standard Joint Vertical Deflection Results

Table 5 shows the deflection results of the analyses considered in Table 4. The vertical deflection at the joint is used as the chief illustrator of the overall behavior of the model under loading. From the data in the table, it is apparent that the model created for this study
overestimates the deflection when compared to the prior researchers’ results. The most likely explanation for this is that, though the rail section properties were matched to the AREMA guidelines for 136-lb rail, the joint bars were not matched to the bars in the published articles. At the joint, since there is a gap in the rail, these joint bars dominate the load transfer mechanism. Therefore, differences in the joint bar moment of inertia will affect the results. Igwemezie and Nguyen (2009) used an orientation in which the joint bars have a “dog bone” cross-section shape with a moment of inertia, $I$, (for two bars combined) of 32.3 in$^4$ while the insulated joint studied by Himebaugh (2006) used joint bars with a combined $I = 24.9$ in$^4$. The total moment of inertia of the two bars in the present model is equal to $(2/12)(1.4”) (4.3”)^3 = 18.56$ in$^4$. Therefore, the shell ABAQUS model has less bending resistance at the joint and it can be expected that more deflection would occur at the joint. Although the agreement of the present model with the Igwemezie and Nguyen (2009a) result is within 10%, the difference is somewhat greater since the tie spacing is smaller in the present model (20” vs. 24”). The tie spacing in the present model matches the spacing of 20” used by Himebaugh (2006).

Table 5. Standard joint vertical deflection results and comparison to prior research.

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Model Max Vertical Deflection (in.)</th>
<th>Reference Deflection (in.)</th>
<th>Percent Difference (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.352</td>
<td>0.323</td>
<td>8.98</td>
</tr>
<tr>
<td>2</td>
<td>0.164</td>
<td>NA</td>
<td>-</td>
</tr>
<tr>
<td>3</td>
<td>0.1568</td>
<td>NA</td>
<td>-</td>
</tr>
<tr>
<td>4</td>
<td>0.1566</td>
<td>0.122</td>
<td>28.36</td>
</tr>
</tbody>
</table>

Additionally, the three models relied on slightly different mechanisms to carry the load at the joint. The standard bolted joint relied on bearing on the joint bar / rail interface and the bolts to carry load across the joint (Igwemezie and Nguyen, 2009a). On the other hand, the bonded insulated joint model transferred all load through the epoxy layer between the rail and joint bars.
(Himebaugh et al. 2008). The model created for the present study utilizes a combination of friction between the joint bar and rail, bearing between the joint bar and rail, and the bolted connection to resist the wheel load. Given the known limitations of the shell model and the differences between the models, the results in Table 5 were considered acceptable and the general behavior of the model matched expectations. Next, the response to loading will be examined in more detail.

### 3.4 Examination of Rail Stresses

The images in Figure 11 show the von Mises stresses around the bolt holes of one of the rail segments when subjected to different combinations of longitudinal and vertical loading. In this case, the foundation stiffness was held constant at 3,000 psi for each loading case to isolate the effect of the load on the response around the bolt holes. In the top image, a 300 kip longitudinal load was applied at the end of each rail to simulate winter contraction of the rails. The bottom image shows the stress contours when the structure was subjected to the 32.5 kip wheel load only.

It is apparent that the very large 300 kip longitudinal tensile load results in a much more critical state of stress around the bolt holes than the condition when only the vertical load is applied. Because the bolts are connected to the holes discretely by the axial elements described in section 2.4, there are large stress concentrations at the discrete connections to the bolt holes. These large stress concentrations close to the hole are judged to be artificial results of this approximate modeling technique. Because the loads are transferred to one point on the bolt and bolt hole, the stresses are higher than they would be if load was transferred surface to surface as the real bolt would bear on the rail bolt hole. Therefore, it is important to direct attention to the stresses somewhat further from the sharp concentrations around the bolts. Looking further from
the bolt holes, it is still apparent that the contribution of the longitudinal tensile load to the von Mises stress is significantly greater than the contribution from the vertical wheel load.

![Figure 11](image.png)

**Figure 11.** Von Mises stress contours around bolts in the web of the right-hand rail. Legend stresses in psi. (a) Vertical load of 32,500 lb and longitudinal load of 300,000 lb. (b) Vertical load of 32,500 lb only.

Figure 11 shows the bolt holes in the web of the right-side rail. As the longitudinal tension force “pulls” the rail to the right the bolts bear on the left side of the holes, resulting in very high stresses on this side of the hole. Although this analysis is elastic, if a rail steel yield stress of 74 ksi is assumed (Igwemezie and Nguyen 2009a), the model indicates extensive localized yielding of the rail steel in these locations. However, this extensive yielding has not been observed in the field, so it is concluded that the shell element model is overestimating the stresses in the rails. It is possible that anchors and clips may dissipate the longitudinal forces in the rails. Therefore, conclusions about the exact state of stress cannot be made, but it can be
concluded that significant tension in the rail during winter months creates a very critical static loading situation in the rails. This reiterates the same conclusions drawn by Igwemezie and Nguyen (2009a). This study also introduces the possibility that these longitudinal loading situations could be a contributing factor in the failure of special track work and diamond crossing bolted connection failure. This will be explored in Chapter 4.
Chapter 4. Diamond Crossing Frog Model

4.1 Overview and Analysis Strategy

This chapter provides a description of the ABAQUS finite element model constructed to investigate the performance of diamond crossings. The strategy and motivation behind the investigation is given and a description of the design of the diamond crossing elements is provided. Once again, shell elements were used to develop relatively efficient models that reasonably represent the behavior of diamond crossings. However, even with this simplification (shell elements instead of solid elements), modeling a crossing diamond introduces significantly more complexity to the analysis compared with the standard joint. In addition, crossing diamonds present appreciably greater design challenges in practice. Whereas bolted joints are fairly uniform—two pieces of rail connected by two joint bars and six bolts—there are countless different diamond crossing designs in place in the field (as detailed in section 1.2.1). Therefore, it was not possible to capture this wide variation in the present study. However, since high-angle crossings create the most critical dynamic loading at a frog, a 90-degree diamond crossing was chosen for the model (Shu and Davis 2008).

As in the standard joint model, shell finite elements were used to model the rails and castings. Since the castings that compose a frog are quite massive and are primarily not composed of relatively thin plate elements, the stresses in these castings will likely not be particularly representative of realistic demands. Though local material deterioration of the point where the wheel tread strikes the frog is certainly an important maintenance issue, it is not the subject of this project. Instead, the focus will be examining the global response of the bolted connections in the diamond since there is no documented data from finite element models or field measurements. As noted above, the variability in diamonds is extensive since the AREMA
diamond crossing plans (shown in Figure 12) are suggested details for solid manganese casting designs, but each manufacturer may have a different variation on this design. Since the detailed drawings of these designs are proprietary, the finite element model developed for this study is intended to represent a generic case that is used in a parametric investigation to study the relative importance of a variety of parameters on diamond crossing bolted connection performance. These parameters are introduced in section 4.5. Different combinations of these parameters were then considered in the finite element model and the response for each case was compared. From this parametric study, the most advantageous or disadvantageous combinations of these parameters were found and discussed. The results of the model are then used to draw conclusions about general procedures that can be implemented in the field. The following section will first describe the design of the elements composing the diamond crossing model.

Figure 12. Plan drawings for a solid manganese steel crossing diamond, showing a number of sections through the diamond (AREMA 2002).
4.2 Crossing Frog Design Development

4.2.1 Casting Design

The design of the crossing model was based on a diamond composed of solid manganese steel castings. The more complicated reversible manganese insert design, which has considerably more individual pieces, was not investigated due to its geometric complexity. A set of AREMA plan drawings for a solid manganese casting diamond is shown in Figure 12 (AREMA 2002). This configuration is composed of four castings, one for each corner of the diamond. Each casting is bolted to two adjacent castings as well as to two rails and two guard rails. This diamond crossing is reflective of the strategy to simplify the diamond geometry and allow for fewer moving parts. Additionally, this is one of the simplest configurations for purposes of computer modeling. A six-bolt connection is used to join the adjacent frog castings, while each casting was connected to the approaching straight rail with three bolts. A short guard rail is also attached at this location with four bolts. Figure 13 shows a photograph of an existing diamond crossing utilizing this general solid manganese casting design, alongside the ABAQUS assembly created to replicate that diamond.

![Figure 13. (a) Photo of the frog in the crossing diamond used for this model (J. Riley Edwards personal communication). (b) ABAQUS model of this frog.](image-url)
Due to the size of the diamond crossing and the uniformity of each corner or quadrant of the diamond, only one casting was modeled fully, which significantly cut down on the size of the model and improved efficiency of the analysis. However, it was also desired to model the inner bolted connections joining the adjacent castings. Therefore, casting “stubs” were created for the adjacent frog castings. These castings were terminated at the point where they reach the frog.

![Diagram of the diamond crossing](image)

**SECTION D-D**

(a) (b)

**Figure 14.** Section cut through bolted connection between two frogs, showing (a) the AREMA (2002) section and (b) the ABAQUS model.

Because of the lack of publications detailing the behavior of diamond crossings in the field and under load, it was difficult to determine the proper properties representing stiffness and moment of inertia. In the end, a generic design was chosen based on the AREMA solid manganese casting plans and sections shown in Figure 12. Instead of attempting to match the section properties of the model design to a certain value, the model sections were designed to be as close to the sections shown in Figure 14 as possible. Additionally, there were a few other guidelines that had to be followed. The distance between the inside of the rail heads in open track is 4’-8½”, which determines the size of the diamond castings (Armstrong 2008). Additionally, because the rail sections developed for the standard joint were 7.0” tall, the height
of the casting sections must all match this height. A section through the casting at the bolted connection between adjacent castings is shown in Figure 14. Features of this section include:

- An approximately 12” wide “foot”
- A “head” that is approximately 8” wide with a flange-way
- Two “webs” that are approximately 1” thick connecting the “foot” and “head”

Figure 14b shows the section created in ABAQUS to model the AREMA section shown on the left. Two of these sections were created, perpendicular to each other, and extracted to create an L-shape to serve as a base for the casting. The flangeway was not explicitly considered in this model.

The shells that compose the castings were linear, reduced-integration shell elements, denoted S4R in ABAQUS. Where bolt holes are cut out of the shells, triangular S3 elements were used to create a finer mesh and more accurately observe the stress transfer between the bolt shanks to the webs of the rail and frog sections.

4.2.2 Rails

The rail segments approaching the crossing frog are exactly the same as those used in the standard joint model. 240”-long pieces of rail are used leading up to the diamond. The section area and moment of inertia (about the horizontal axis) are matched to the AREMA suggestions for 136-lb rail (AREMA 2010). Although the section created in 2.3.1 has a lateral moment of inertia of 20.3 in\(^4\), approximately 1.4 times that of the AREMA (2010) recommended section lateral \(I = 14.4\) in\(^4\), the lateral moment of inertia of the rail is assumed to have little influence on the general behavior of the bolted connections near the diamond, which is much stiffer and more
robust. Because this rail section is held constant for all analysis cases, its lateral stiffness was deemed to have little to no effect on the parametric study.

As the train approaches the diamond, there is a guard rail present to help with alignment and prevent the rail wheels from hunting laterally as they cross the diamond. The dimensions of the guard rail are the same as the open track rail, with one exception. Part of the foot of the guard rail is cut off. Because spacing between the web of the running rail and guard rail is 4” in the ABAQUS model and the foot of the rail is 6” wide, 2” of the guard rail foot is cut off on one side, as shown in Figure 15. Both the rail and guard rail were meshed using S4R shell elements except around bolt holes, where S3 finite elements were used.

![Figure 15. (a) AREMA section cut through the bolted connection between guard rail, rail, and casting (AREMA 2002). (b) Section view of ABAQUS assembly of these same elements.](image)

**4.2.3 Bolts**

The bolts were created in the same manner as for the standard joint model, with the exception that there are three different bolt lengths in the diamond crossing model. As the open track approaches the diamond, it first encounters the guard rail. Connecting the guard rail and running rail is one bolt, 4.6”-long from the inside of the nut to the inside of the head, flush with the guard rail and open track. This bolt is denoted by (a) in Figure 16. Moving through the diamond, the first connection between the guard rail, straight rail, and frog is encountered,
indicated by (b) in Figure 16. Here, the bolt shank is 7.3” in length between the nut and the head. There are three bolts of this length forming this part of the connection before the actual frog is encountered. After the first frog is passed, the next connection region is the internal connection between frogs (location (c) in Figure 16). Here, six 10”-long bolts pass through two joint bars and the two casting webs to form the connection between adjacent castings. B31 linear beam elements are used for the bolt shanks and S3 triangular shell elements are used to model the bolt nut/head. The temperature differential introduced in 2.5.2.1 was applied to the bolt shanks in order to impose a pre-tension force of nearly 30,000 lbs.

Figure 16. Plan view of the diamond crossing assembly, showing bolt locations.
4.2.4 Joint Bars

In the crossing diamond design studied, there is a six-bolt connection that is fairly similar to the six-bolt connection used to connect two pieces of rail in a standard joint. However, in this case the six-bolt connection joins two adjacent frogs. Therefore, at a single diamond, there would be four six-bolt connections. A section cut through this connection is shown in Figure 14. The figure shows 2.5-in. thick, 29”-long joint bars connecting the two frogs. For simplicity, the ABAQUS model used 30”-long bars. As in the standard joint, the bars are composed of a single plane of S4R and S3 shell elements. These elements are formulated for large thicknesses, which is the case for all of the crossing frog sections. Once again, 1”-diameter bolts were used in the connection, with 1 1/8”-diameter holes. As before, the bars are meshed with predominantly 4-sided elements, with triangular elements used around the bolt holes. The AREMA plans indicate that these bolts are not uniformly spaced at the joint (2002). There is a 4 ½” spacing (on center) between bolts that pass through the same frog casting, while the last bolts in each frog casting are spaced 6” on center.

4.2.5 Spacer Bars

As the AREMA plans show (Figure 15), there is a casting set between the guard rail and open track rail to maintain the proper spacing between the two (this is denoted with the term “spacer bar” for the rest of this paper). The spacer bar was also modeled as a plate meshed with shell elements. The behavior of this bar was not of particular interest, so it was meshed more coarsely than other parts of the model. It is defined in contact with both the guard and mainline rail webs so that shear and normal force is transferred between these parts of the model.

The effects of including similar spacer bars at other locations in the model will also be observed. Between the two “webs” of the frog casting, there may be a space where the bolt
“floats.” In some cases, the casting manufacturer may include a “shroud” over the bolt in these locations in order to make casting easier. However, it was apparent that this is not always the case after visiting a manufacturer’s casting facility. Therefore, a spacer bar was introduced between the “webs” of the frog casting to provide a more direct load path for the transfer of bolt preload. These spacer bars were assumed to have wide enough holes that they will not bear on the bolts and thus will not transfer any transverse loading to the bolts.

4.2.6 Base Plate

One significant difference between the support conditions of the standard joint and those of a diamond crossing is the presence of a thick steel base plate supporting the diamond. Often, this base plate has a seat milled into it to provide additional restraint to the diamond casting elements. A range of base plate thicknesses were considered for the model and their effects on the model response are discussed in Chapter 5.

4.3 Material Properties

The materials used were the same as described in 2.2. Rail steel with elastic modulus of 30,000 ksi was used for all elements of the diamond crossing except the stiffened bolt head and nut (see section 2.3.3) and the truss and beam connectors (see 2.4).

4.4 Assembly and Contact Definitions

Hard contact between steel surfaces was defined in ABAQUS using the penalty method and a friction coefficient of 0.35. Contact was defined between all surfaces which were flush with each other. For example, the base plate supports the castings through contact only, and no other connectors were used. Node to surface contact definitions were used for top and bottom edges of bars and plates that need to bear on the foot or head of the rails or castings.
4.5 Loads, Supports, and Boundary Conditions

4.5.1 Applied Loads

Bolt tension was applied to all bolts in the same manner as described in the standard joint. Bolt tensioning was carried out in two stages—one for bolts in each orthogonal direction—to decrease analysis times. Longitudinal loads were examined in some cases and they were applied as concentrated forces at the free rail ends. A range of longitudinal loads was examined and in most cases, a value of 100 kips was used. A dynamic amplification factor of 5 was used to multiply the vertical wheel load from 44 kips to 220 kips. This factor was drawn from the plots in Figure 3. The high end of this range would be considered a very extreme scenario for a new diamond, but Shu and Davis (2008) indicate that equivalent wheel loads could be significantly higher on already-worn diamonds.

4.5.2 Elastic Foundation

The elastic foundation was again used to support the base plate and rails at tie locations, using a base value of $k = 3000$ psi for the equivalent beam on elastic foundation. To convert this foundation stiffness for a beam on elastic foundation to a three-dimensional value, Eqn. 1 was used. For the diamond crossing, the tie spacing remains at 20,” but the ties are all 10” wide. Therefore, Eqn. 1 results in an elastic foundation value of 1000 lbs/in/in². Where the crossing frog is supported by a baseplate, the required elastic foundation stiffness is recalculated. It is assumed that ties directly support the entire base plate beneath the diamond. The base plate is 16” wide. Computing the foundation stiffness as before:

$$k_{eff} = \frac{k \times s}{A_{tie}} = \frac{(3 \text{ ksi})(1”)}{(16”)(1”)} = 0.1875 \text{ kip/in/in}^2$$
Since the area of support is much larger, the foundation stiffness used is significantly smaller. This value can be easily re-calculated for any base foundation stiffness $k$ using the above formula.

4.5.3 Boundary Conditions

Boundary conditions were applied at three different locations within the model. Two of these locations occur at the far end of each rail. Once again, the longitudinal axis of each rail was left free with all other degrees-of-freedom fixed. Additionally, there is a symmetry condition at the casting-to-casting connection perpendicular to the direction of travel, shown at section A in Figure 17. Because the model represents a casting in a 90-degree crossing, both wheels on the same axle will strike adjacent frogs at nearly exactly the same time when a train passes over the diamond traveling in the Y-direction. Therefore, at this section, degrees-of-freedom 1, 5, and 6 were restrained, while degrees-of-freedom 2, 3, and 4 were left free.

Figure 17. Plan view of diamond casting assembly, showing the section at which the symmetry boundary condition is applied.
Finally, the boundary conditions at the ends of the stubs were considered, where stiffness is provided by the adjacent castings and the rest of the diamond. At the end of the stub connecting to the symmetry boundary condition, a constraint was used to tie the degrees-of-freedom at the end of that stub to the corresponding section in the casting studied. For the other end, however, a new connector was developed. This connector joined the centroid of the stub casting section to the ground and its stiffness was input directly. In order to determine this stiffness matrix, a modified model was created, cutting out one of the casting stubs and part of the main casting, up until it frames into the frog at the edge of the guard rail. Unit deflections and rotations were imposed on the model at this cut-off location. The reaction forces were then measured and used to create a stiffness matrix that could be applied to the ground-to-stub connectors to represent the effects of the adjacent frogs.

4.5.4 Structural Mesh

The mesh used to perform the analysis of the crossing frog is shown in Figure 18. The mesh contains 124,937 elements and 119,335 nodes. Linear elastic analyses performed with ABAQUS/Standard using this mesh were completed in approximately 50 minutes using an 8-processor computing cluster.

![Figure 18. Meshed crossing frog assembly, showing close-up of frog region.](image)
4.6 Parametric Investigation

This section will list a number of parameters and diamond properties that were varied. Different combinations of these parameters were used within the structure, and the results were examined. The results of each permutation were then compared to determine the relative contribution of each parameter on the behavior at the bolts. The effects of the following were investigated:

- Vertical load position
- Magnitude of longitudinal load
- Load combination
- Lateral and longitudinal spring restraint placed on rails
- Effect of the spacer bars
- Base plate thickness
- Elastic foundation value
- Bolt preload

The chief measure of the effects of these parameters was bolt stresses, which were plotted and compared. The parametric investigation will be discussed further in Chapter 5.
Chapter 5. Diamond Crossing Frog Analysis Results

The following sections investigate the response of the 90-degree crossing frog model under a variety of loading combinations. The effects of internal spacer bars, loading, foundation stiffness, base plate thickness, and lateral and longitudinal restraint are considered. In addition, a potential modest modification, where the guard rail leading into the frog is integrated with the frog, is studied.

5.1 Effect of the Spacer Bar on Bolt Tensioning

As described earlier, the spacer bar refers to the additional element internal to the frog casting that has been examined as a potential approach to maintain continuity of contact through the bolted connections. This section details the effect of this spacer on the bolt pretension. Because Ramey and Jenkins (1995) indicated that maintaining bolt preload is important in preventing loosening of bolted joints under dynamic vibration loading, maintaining the specified AREMA bolt pretension was judged to be an important parameter in preventing special track work joint loosening and damage. The effect of this spacer on the bolt tension force was considered under the pre-tensioning stage, before any external load was applied to the model. The spacer bars were then employed in the model for analysis of all successive loading stages. In the following sections, the influence of bolt tension on bolt shear stresses under loading will be considered. In reality, the pretension may be reduced during service such that there is slip at the faying surfaces and the bolts go into bearing at the holes. Due to the modeling technique (using truss elements to connect the bolts to the bolt holes) some level of bearing is always occurring in the model even when the bolts are fully pre-tensioned.
Table 6. Preload results in the 7.3” and 10” bolts, with and without spacer bars.

<table>
<thead>
<tr>
<th>Pre-Tension (kips)</th>
<th>Percent Increase (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case</td>
<td>7” Bolt</td>
</tr>
<tr>
<td>No Spacers</td>
<td>22.56</td>
</tr>
<tr>
<td>Spacers</td>
<td>28.09</td>
</tr>
</tbody>
</table>

The addition of spacers was studied in the two regions described in the model formation section of the paper. Table 6 shows the bolt axial force after the pre-tension stage for a 10” bolt at the joint between frogs and for a 7” bolt at the joint where the rail is bolted to the frog. In both of these cases, the target preload was 30,000 lbs, and cases with and without spacers were investigated. The table shows that the spacer bars do increase the retention of bolt pre-load. The spacer bars increase stiffness through the bolted region and maintain a snug fit between all elements of the bolted connection, thus preventing local bending of the frog webs or joint bar elements. Focusing on the 10” bolts at the connection between frogs, the preload is reduced by local bending of the webs and joint bars. Figure 19 shows a section through this connection. When there is no spacer, and the fit is not snug through the entire connection, the pre-tension forces must flow around the frog section. When the spacer is included in the connection, the forces are able to flow directly through the connection, and the contact stresses between the connection elements are highest around the bolt holes.

![Figure 19](image.png)
Von Mises and contact stress contours in the joint bars are shown in Figure 20. This figure shows one of the 2.5”-thick joint bars joining the two adjacent frog pieces. The von Mises stresses indicate a low degree of out-of-plane bending resulting in stresses in the range of 3-5 ksi. Additionally, the contact stress contours indicate that, while some of the contact occurs around the bolt holes, much of the transfer of stresses occurs at the top and bottom of the joint bar, where the web of the frog is stiffened by its “foot” or “head.” Figure 21 shows the contact pressure contours in the joint bar when the spacer bars were included in the model. When the spacers were added, the normal force resulting from the pre-tensioning of the bolt is transferred via contact stresses in the region immediately surrounding the bolt holes and local bending stresses in the joint bar are essentially eliminated.

Figure 20. Joint bar (a) von Mises and (b) contact stress contours (legend stresses in psi) for case without spacers.
5.2 Bolted Connection Results

Figure 22 shows the location of the 7.3”-long bolt examined in the following sections, highlighted in red. This bolt will be referred to as Bolt A throughout the rest of the text. It was selected because it passes through the frog, guard rail, and running rail. Additionally, this bolt passes through the rail that the wheel would be traveling on in the loading case considered. Although it was not necessarily the most critical for every combination of loading, all of the 7.3”-long bolts behaved similarly for the cases studied. Thus, the relative behavior between the different conditions studied is judged to be similar for all of the 7.3”-long bolts.

Figure 22. (a) Plan of crossing frog with highlighted 7” bolt studied in section 5.2. (b) Three-dimensional view of frog with same bolt highlighted. The white arrows show the location of the wheel load.
Once the bolt of interest was chosen, the truss connections around this bolt were refined in the same way as for the standard joint model described in section 2.4. In this case, the radial truss members were spaced 90-degrees apart, with 3 sets of radial truss members per bolt hole. These radial truss members were connected to a stiff beam element before framing directly into a point on the bolt hole. The bolt shear forces, shear stresses, and bending stresses were then investigated when the model was subjected to a variety of different combinations of loading. Additionally, the 4.6” bolt marked in Figure 22 was investigated in some cases and will be referred to as Bolt B.

The maximum stresses in the bolt shank at a given section were determined using the forces and moments acting on that section. In ABAQUS, section forces SF1, SF2, SF3 and section moments SM1 and SM2 were found at points along the bolt shank. To calculate stresses, the following formulas were used:

\[
Axial\ tensile\ stress\ =\ \sigma = \frac{SF1}{A} \tag{11}
\]

\[
Maximum\ shear\ stress\ =\ \tau = \frac{4\sqrt{(SF2)^2 + (SF3)^2}}{3A} \tag{12}
\]

\[
Maximum\ bending\ stress\ =\ f = \frac{r\sqrt{(SM1)^2 + (SM2)^2}}{I} \tag{13}
\]

In these formulas, \(A\) is the bolt area, \(r\) is the bolt radius, and \(I\) is the moment of inertia of the bolt shank section. To determine the maximum axial tension stress in the bolt, Equations (11) and (13) are added. These formulas were used to determine the stresses at each section, evaluate the variation along the length of the bolt (as shown in Figure 24) and identify the maximum stress condition.
Figure 23. Diagram of bolt and truss connectors that join the webs of the rails and frog sections.

Figure 24. Example bolt shear stress plot when model subjected to thermal tension from rails. The figure shows the location of the rail, guard rail, and frogs that connect or “bear” on this bolt.

5.2.1 Effect of Preload

Ramey and Jenkins (1995) demonstrated the importance of maintaining bolt preload (or bolt tension) to prevent loosening of bolted connections under vibrations. However, bolt preload is also very important even for static loading. Because the bolt tensioning results in a clamping force between the contact surfaces in the diamond crossing bolted connections, this allows a larger portion of load to be transferred through friction, thus reducing the stresses in the bolts. This is summarized in Table 7.
Table 7. Summary of maximum stresses at any section through the bolt, showing the importance of bolt preload (tension) in reducing stresses due to loading.

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Bolt Preload</th>
<th>Max Bolt Shear Stress</th>
<th>Max Bolt Bending Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 kips</td>
<td>0 kips</td>
<td>29.6 ksi</td>
<td>110.3 ksi</td>
</tr>
<tr>
<td>longitudinal</td>
<td>28.1 kips</td>
<td>21.6 ksi</td>
<td>98.7 ksi</td>
</tr>
<tr>
<td>220 kips</td>
<td>0 kips</td>
<td>4.6 ksi</td>
<td>29.6 ksi</td>
</tr>
<tr>
<td>vertical</td>
<td>28.1 kips</td>
<td>2.9 ksi</td>
<td>21.6 ksi</td>
</tr>
</tbody>
</table>

5.2.2 Effect of Loading

The results from three loading scenarios are summarized in Table 8, and the corresponding von Mises contour plots are shown in Figure 25. The bolt examined in Table 8 bears on the central hole in Figure 25. In these three cases, all parameters except load combination were held constant, thereby isolating the effect of load. The vertical load case represents a relaxed situation, with no thermal shrinking of the rail entering the diamond. The longitudinal case represents a winter situation when all anchoring of the rail as it approaches the diamond has failed, and no train is passing over the diamond. The longitudinal-plus-vertical case considers the situation where a train passes over the diamond during the winter months, and there is no longitudinal anchorage. Figure 25 shows contour plots of the von Mises stress around the bolt holes in the rail that frames into the diamond at this connection. These images show that the longitudinal load can result in very high stresses around the bolt holes. The gray regions in the figure show areas where the stress exceeds 30 ksi (a value chosen for consistency between images and so that lower levels of stress variation away from the high-stress regions are visible). The images also show that the vertical wheel load alone does not seem to cause particularly high stresses near the bolt holes. It is recognized that, although a large dynamic amplification factor is used to approximate the increase in load above the static wheel load magnitude, the intricacies of the impact loading scenario are not fully captured. However, the 100 kip longitudinal rail
A tensile load caused a very large spike in the stress around the bolt holes, where localized stresses exceed typical rail steel yield stress, which is around 74 ksi (Igwemezie and Nguyen 2009a).

Figure 25. Contour images of von Mises stress around bolt holes in the rail framing into the diamond crossing (legend stresses in psi). (a) Vertical load of 220 kips only. (b) Longitudinal load of 100 kips only. (c) Longitudinal load (100 kips) and vertical load (220 kips).

From the results in Table 8, it is clear that the longitudinal load that enters the diamond from the rail has a significant effect on the shear stress of the bolts in the rail-to-diamond connection. The 100 kip longitudinal load dominates the shear stresses when compared to the 220 kip vertical load. This is likely due to the fact that the bolted connection’s flexural resistance is significantly higher than its resistance to longitudinal (axial) rail loads. Additionally, the vertical wheel load is redistributed through the casting elements and into the foundation without affecting the bolted connection, whereas the entire longitudinal load is transferred through the rail to the bolted connection. However, the application of the static,
amplified wheel load when the longitudinal load was already present actually reduced the shear stresses in the bolt. The longitudinal loading case causes bolt shear forces that occur in the horizontal plane. The vertical wheel load causes vertical and horizontal shear force components in the bolts, but the horizontal shear force occurs in the opposite direction as that due to the longitudinal load. Thus, the combined vertical and longitudinal loading reduces bolt shear stresses.

Table 8. Bolt shear stresses under three different loading conditions.

<table>
<thead>
<tr>
<th>Base Plate Thickness</th>
<th>Foundation Stiffness</th>
<th>Vertical Wheel Load</th>
<th>Longitudinal Rail Load</th>
<th>Max Bolt Shear Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5&quot;</td>
<td>3 ksi</td>
<td>220 kips</td>
<td>0 kips</td>
<td><strong>2.9 ksi</strong></td>
</tr>
<tr>
<td>1.5&quot;</td>
<td>3 ksi</td>
<td>0 kips</td>
<td>100 kips</td>
<td><strong>21.6 ksi</strong></td>
</tr>
<tr>
<td>1.5&quot;</td>
<td>3 ksi</td>
<td>220 kips</td>
<td>100 kips</td>
<td><strong>10.9 ksi</strong></td>
</tr>
</tbody>
</table>

The relative contributions of the vertical and longitudinal loading do not seem consistent with field observations. However, there is no data to use for model validation, so in the absence of guidance for further refining the model, it is used as currently constructed to make further observations about relative importance of various parameters. The following issues are noted as possible aspects of the model that contribute to the unexpected relative contribution of vertical and longitudinal loading:

1. The stiffness of the model may be too large, thus decreasing the influence of the vertical load.
2. The actual longitudinal thermal load entering the connection may be much smaller than applied in the model.
3. Dynamic, impact, and vibration effects of the wheel loading may not be captured reasonably by the static, linear analysis.
Quantifying the state of stress and deformation in diamond crossings under loading is a very important area for future research.

5.2.3 Load Case: Longitudinal Tension Only

5.2.3.1 Effect of Longitudinal and Lateral Restraint

Table 9 shows maximum bolt shear stress along the shank of the bolt when the model was subjected to thermal longitudinal loads in the rails only. In these cases, a tensile load of 100,000 lbs was applied to both rails framing into the joint. The maximum bolt shear stress occurs in the region where the bolt bears on the rail bolt hole.

Table 9. Effect of rail anchorage on shear stresses in Bolt A.

<table>
<thead>
<tr>
<th>Anchors (Springs)</th>
<th>Max Bolt Shear Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>none</td>
<td>21.2 ksi</td>
</tr>
<tr>
<td>lateral</td>
<td>21.6 ksi</td>
</tr>
<tr>
<td>longitudinal</td>
<td>13.1 ksi</td>
</tr>
</tbody>
</table>

The effect of lateral restraint is studied by applying stiff springs connecting the foot of the rail to the ground at five discrete locations. These springs were given a stiffness of 100 kips/in, and they prevent significant out-of-plane bending of the rails that arises due to model asymmetry. The shear stress values in Table 9 show that the presence of the lateral restraint has a negligible impact on the bolt stresses, and causes the bolt stresses to increase slightly. When the lateral restraint is replaced with springs that act in the longitudinal direction of the rails, the bolt stresses are decreased significantly. The longitudinal restraint models the effects of the anchors that are applied to prevent creep of the rail (AREMA 2010). In the case shown, the longitudinal springs have a stiffness of 100 kips/in. Because the analysis assumes linear elastic behavior of all parts of the model, it is assumed that the anchors will not reach their capacity.
under the applied loading. The addition of many of these anchors (one at each of the last five ties approaching the diamond) significantly decreased the bolt shear stress. This shows the importance of a longitudinal anchoring system to take the tension out of the rails before they are bolted to the diamond. Although lateral and longitudinal restraints were considered independently, in reality the rail anchors will provide restraint to both simultaneously. This combined restraint case was also studied, but found to be almost identical to the longitudinal anchor case. Although the effect of longitudinal restraint is potentially quite significant, no field data is available for comparison, and industry opinion varies widely as to the degree that anchors reduce longitudinal forces in diamonds.

![Figure 26. Connection between the frog (left) and guard rail (front, right). The connector springs are highlighted in red.](image)

5.2.3.2 Effect of Tying Guard Rail and Diamond Casting

Next, the connection of the guard rail to the frog was examined at the location highlighted in Figure 26. In the case of a standard joint, two rails may bear on one another via an end post that maintains the separation between the rails but does not provide any resistance to tension in the joint. Similarly, in a crossing diamond, it appears that the straight rail and guard
rail are allowed to bear on the diamond at the joints, as shown in Figure 27, but only the bolted connection provides resistance to tensile loads. However, there is only one bolt passing through both the frog and guard rail, and there is a gap at the joint between the guard rail and the main portion of the frog. Therefore, there is not a direct path for the longitudinal load to flow into the frog from the guard rail. A case in which the guard rail is essentially joined to the frog at the location shown in Figure 26 was examined. In this case, the guard rail and frog were connected by very stiff springs with stiffness $k = 1 \times 10^7$ ksi restraining motion in the rail axis direction. This creates a connection that allows the bolts to act in double-shear, dividing the load more evenly to both sides of the connection, as shown in Figure 28a.

Figure 27. Close-up of guard rail bolted connection to crossing diamond (J. Riley Edwards personal communication 2010).

Figure 28 shows diagrams of the horizontal shear force component along Bolts A and B at the rail-to-frog connection. The blue curve shows the horizontal shear force in the bolt in the case where the guard rail was only attached to the frog through the last bolt of the connection. When the nearly-rigid springs were used to attach the frog to the end of the guard rail, the shear force on Bolt A as it bears on the frog levels off to approximately the same force as is seen in the left side before it bears on the guard rail. As noted above, the “spacer” between the straight rail
and guard rail was assumed to have a wide enough hole so that the bolt will not bear on the spacer. This is reflected in the long segment of constant shear force in the range \( x = [-2.7, 0.7] \).

This new double-shear connection serves to reduce the maximum shear forces in Bolt A and the other 7” bolts and—as Figure 29 shows—the stress around the bolt holes. The force is
distributed to Bolt B, which previously carried very little load. Though Bolt B is depended on to carry more load, the shear force in this bolt is still less than the force in Bolt A. Though connecting the guard rail to the diamond casting does improve performance, it is not as effective as the longitudinal springs. This is because the longitudinal springs remove some of the longitudinal load before it reaches the diamond. However, Figure 29 does show that a combination of these two methods discussed in 5.2.2.1 and 5.2.2.2 can work together to further decrease the state of stress around the bolt holes due to longitudinal thermal stresses in the rails.

![Diagram](image)

Figure 29. von Mises stress contours near rail bolt holes for longitudinal load of 100 kips (legend stresses in psi). The longitudinal force pulls the rail to the left in these images.

### 5.2.4 Load Case: Vertical Load Only

The following results describe the response of the model to a “summer” loading case, in which the rails are “relaxed” and carry no longitudinal force. Because parameters including the
base plate thickness and foundation stiffness most significantly affect the response under vertical loading, their influence on behavior will be examined in this section.

5.2.4.1 General Behavior

Under typical summer conditions in which only the amplified wheel load was applied, the maximum vertical deflection of the crossing diamond was 0.396 in. The lack of published field measurement data means that the accuracy of this deflection cannot be verified. However, it is possible that the dynamic effects resulting from the impact made by the wheels repeatedly impacting the casting could cause higher deflections. Compared to the standard joint, the deflections are higher even though the frog has a significantly higher modulus. Therefore, even in the static analysis the amplified wheel load is enough to overcome the higher stiffness. A rendering of the crossing frog deflection under vertical loading is shown in Figure 30. The image shows the deformation amplified 25 times. This loading condition ignores the presence of other wheels on the train as it is the result of a single axle traveling across the diamond. The direction of travel in this case would be from the lower left part of the image to the upper right. Because this is a frog in a 90-degree crossing, the deflection of the ends of the two adjacent frog “stubs” is different. The unmodeled adjacent frog to the upper left would deflect the same amount as the frog studied because both wheels on the same axle would “jump the gap” at the same time. This symmetry condition was explained previously in section 4.4.2.
Figure 30. ABAQUS deformation rendering of crossing frog under amplified vertical loading (25x amplification).

The model response reflects this symmetry-of-loading condition, and the results show that the bolted connection between these frogs tends to deflect downward as a unit, providing little restraint to vertical deformations. Therefore, the von Mises stresses in the plates and bolts at this connection are not critical, as Figure 31 shows. The maximum stresses in the contour plot in Figure 31 are centered in the bolt shank and the location where the bolt shank meets the bolt nut and head, which arise from pre-tensioning of the bolt. The shear force and moments within these bolts are very small, indicating that the pre-tensioning still dominates the stresses in these bolts.

Figure 31. Von Mises stress (psi) contours at internal casting-to-casting connection perpendicular to direction of travel.
5.2.4.2 Position of Vertical Wheel Load

The effect of the vertical wheel load position was also examined. The vertical load was positioned at three different points on the rail or frog and four different analyses were conducted to determine the most critical position for the bolt shear stress. The effect of preceding or trailing wheels was not considered. Figure 32 shows the locations of the three wheel loads considered. Load (1) considers the nominal wheel load of 44 kips when placed directly above the bolt examined and highlighted in Figure 22. Load (2) is the case where the 44 kip load was placed at the joint between the end of the rail framing into the diamond and the diamond casting section. Load (3) corresponds to the previously examined 220 kip vertical load, using the dynamic amplification, placed at the flangeway gap. Load (4) considers the nominal 44 kip wheel load placed at the gap location as in loading case (3); this condition would correspond to the case of flange-bearing frogs in which there is little to no dynamic amplification of the vertical wheel load. As expected, the 220 kip load results in the greatest shear stresses in the bolt even though it was not placed directly above the bolt investigated. However, the shear stresses in the bolt when subjected to load case (4) are lower than from cases (1) and (2), showing that very “quiet” flange-bearing frogs may shift the critical wheel loading location completely. Because the central casting section is much stiffer (in other words, the central part of the diamond has a higher moment of inertia), it provides greater resistance to the static wheel loads than the regions of the bolted connection between casting, guard rail, and running rail. Therefore, it is apparent that there is a very significant advantage to the flange-bearing frogs in not only reducing bolt stresses, but transferring the critical wheel loading location.
Table 10. Maximum bolt shear stresses for the different vertical loading situations shown in Figure 32.

<table>
<thead>
<tr>
<th>Vertical Load Location</th>
<th>Vertical Wheel Load</th>
<th>Max Bolt Shear Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>44 kips</td>
<td>622 psi</td>
</tr>
<tr>
<td>2</td>
<td>44 kips</td>
<td>529 psi</td>
</tr>
<tr>
<td>3</td>
<td>220 kips</td>
<td>2864 psi</td>
</tr>
<tr>
<td>4</td>
<td>44 kips</td>
<td>305 psi</td>
</tr>
</tbody>
</table>

5.2.4.3 Effect of Base Plate Thickness

Base plates of two different thicknesses were tested while maintaining a constant elastic foundation stiffness of 3 ksi. For many diamonds, a seat is milled out of the base plate where the rails must pass. Thus, there are two different thicknesses in the plate. However, this analysis considers only the case where the entire base plate is the same thickness as the rail seat.

Bolt A, highlighted in Figure 22, was again investigated and the results are shown in Table 11. The table shows that doubling the thickness of the base plate from 0.75” to 1.5” results in very little change in bolt shear stresses. At the location of maximum shear stress—where the bolt “bears” on the rail—the shear stress is decreased very slightly when using the thicker base plate. On the other hand, the maximum axial stresses actually seem to be slightly increased. In this case, the axial stress in the bolt was equal to 35.8 ksi before the wheel load...
was applied. The application of the vertical wheel load introduced fairly low levels of bending stress, increasing the maximum axial bolt stress (due to bending and tension) to nearly 45 ksi. Considering tension-shear interaction, these stresses are still in the elastic range using a rail steel yield stress of 74 ksi as suggested by Igwemezie and Nguyen (2009a). Additionally, the differences in shear and axial stresses, due to the approximations made in the model, are essentially negligible. In the case of the shear stress, the slight improvement with the doubling of base plate thickness was likely due to the slight additional stiffness added to the model by the thicker plate section.

Additionally, Table 11 shows that the improvement in vertical deflections is very small when using the thicker base plate. It must be concluded that the static analysis indicates that only a very slight improvement in behavior can be made by using larger base plate sizes. Due to the price of material, it is likely that this improvement would be negated by cost. It is recommended that dynamic analysis be performed on a more sophisticated model to determine conclusively the effect of base plate thickness on diamond crossing behavior. However it is suggested that the base plate does not have much of an effect on diamond crossing performance because it would require a significant thickness increase to impact the overall diamond stiffness.

Table 11. Maximum frog deflections and bolt shear stresses for the base plate thicknesses considered.

<table>
<thead>
<tr>
<th>Base Plate Thickness</th>
<th>Max Vertical Deflection</th>
<th>Max Bolt Shear Stress</th>
<th>Max Bolt Axial Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.75”</td>
<td>0.402</td>
<td>2.65 ksi</td>
<td>44.2 ksi</td>
</tr>
<tr>
<td>1.5”</td>
<td>0.396</td>
<td>2.57 ksi</td>
<td>44.5 ksi</td>
</tr>
</tbody>
</table>

5.2.4.4 Effect of Foundation Stiffness

A range of elastic foundations stiffness values were tested while a constant base plate thickness of 1.5” was maintained. Because the elastic foundation acts normal to the face on
which it is applied and the longitudinal load results in very minimal vertical deformations, the
effect of foundation stiffness was examined under vertical load only. Foundation stiffness values
were chosen based on the equivalent beam-on-elastic-foundation values and then converted to
the ABAQUS values (in lbs/in/in²) as described in section 2.5.1. The original 2-D beam on
elastic foundation stiffness values are shown with the results in Table 12. The results show that
foundation stiffness has a much more significant effect on the bolt stresses and global deflection
of the frog compared to the base plate size. Because this was a linear elastic analysis and the
entire vertical load is eventually transferred to the elastic foundation, these results reflected
expectations. Thus, halving the foundation stiffness resulted in a large spike in shear and
bending stress in the bolt because deflection and flexing of the frog increased significantly. On
the other hand, increasing the foundation stiffness by 50% resulted in very attractive results. The
bending stresses were significantly reduced to the point where they accounted for only about
10% of the peak axial stress (which was approximately 36 ksi due to bolt preload).

<table>
<thead>
<tr>
<th>Foundation Stiffness</th>
<th>Max Vertical Deflection</th>
<th>Max Shear Stress</th>
<th>Max Axial Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5 ksi</td>
<td>0.6562”</td>
<td>5.1 ksi</td>
<td>49.9 ksi</td>
</tr>
<tr>
<td>3 ksi</td>
<td>0.4035”</td>
<td>2.9 ksi</td>
<td>44.2 ksi</td>
</tr>
<tr>
<td>4.5 ksi</td>
<td>0.2982”</td>
<td>1.9 ksi</td>
<td>41.8 ksi</td>
</tr>
<tr>
<td>6 ksi</td>
<td>0.2401”</td>
<td>1.4 ksi</td>
<td>40.4 ksi</td>
</tr>
</tbody>
</table>

Table 12. Summary of foundation stiffness investigation.

Figure 33 is a plot of the maximum bolt stress as a function of the elastic foundation
value. The plot shows both the maximum axial stress due to bending and bolt tension (preload),
and shear stress for each of the four elastic foundations tested. The results indicate that
increasing the foundation stiffness helps significantly in reducing bending and axial stresses
when the initial foundation stiffness is low. However, this advantage seems to progressively
decrease as the foundation stiffness parameter continues to increase. Additionally, Table 12
indicates that the decrease in vertical deflections begins to level off as the foundation stiffness exceeds 4.5 ksi.

Figure 33. Plot of maximum stress in Bolt A versus elastic foundation.
Chapter 6. Summary, Conclusions, and Recommendations for Future Work

This research studied bolted connections in railroad diamond crossings to better understand behavior and load transfer mechanisms. These bolted connections are high maintenance components of special track work, and even modest improvements have the potential to provide cost savings to the rail industry. Shell elements were used to create simplified models of railroad bolted connections for use in finite element analyses. An investigation into the behavior of standard bolted rail joints was successful in confirming the critical stress locations observed in the field and in prior research. The standard rail joint model provided initial insight into the behavior of bolted connections diamond crossings. Finite element analyses of a diamond crossing frog and its connections were used to test hypotheses developed after the literature review and standard joint analysis. In this chapter, conclusions are presented and recommendations for future research are discussed.

6.1 Conclusions

Relatively simple models of diamond crossings were developed for static analysis. Although there is no published data on diamond crossing behavior to use for model validation, valuable information was gleaned from this study by making relative comparisons between cases that demonstrate the effect of different parameters. This study is a first step towards a deeper understanding of special track work bolted connection behavior. However, field monitoring is a critical future activity for investigation of bolted connections in special track work.

As a starting point for this research, a simple rail joint was modeled in order to provide insight into the behavior of bolted rail joints. From the standard joint model it was concluded that: the results from the shell element model matched the expected behavior of straight rail and
a standard joint reasonably well, based on comparisons with prior numerical simulations and analytical beam on elastic foundation theory; when applied, longitudinal thermal tensile loading dominates the stresses in bolts connected elements. Since the simplified modeling approach using shell, beam and connector elements proved reasonable for the standard joint, it was extended for use in diamond crossings. Due to the importance of longitudinal loading in standard rail bolted joints, their effect in diamond crossings was also considered. Though insulated joints and rail anchors surround all diamond crossings, it is not clear how effective they are for dissipating thermal stresses.

Based on the parametric variations that were considered in the present study, the following major conclusions can be drawn:

- Maintaining bolt preload is important since it allows forces through the connection to be transferred by friction, which reduces shear on the bolts and bearing on the connected elements. Prior research (Ramey and Jenkins 1995) demonstrated the importance of maintaining bolt preload in connections subjected to vibration cycles.
- Introduction of an internal spacer or bolt sleeve internal to the frog was beneficial for reducing local deformation in the connected elements and maintaining bolt pretension.
- Just as in the standard joint, longitudinal loads in the bolted frog connections had a very significant effect on the bolt and connected element stresses. Large thermal longitudinal loads, with magnitudes based on recommendations for open track, produced demands that were much more significant than demands from the vertical load. It is usually assumed that the dynamic loading situation arising from passage of wheels over the flangeway gap is the driving force behind diamond crossing failures, but the present research shows that longitudinal loads in the rails may also be a factor. The finite element results confirmed the
importance of an anchoring system in removing thermal longitudinal load in the region before the rails are bolted to the diamond crossing.

- Mobilizing double-shear bolt behavior between the rail and diamond crossing through continuity at the guard rail helped reduce bolt shank shear stresses.
- The foundation stiffness and base plate stiffness primarily influenced the diamond crossing frog behavior when subjected to the amplified vertical wheel loads. As expected, these parameters had very little influence on the behavior of the frog when subjected to the longitudinal thermal loads. Increasing the thickness of the base plate stiffened the entire structure slightly, but resulted in a negligible change in bolt performance. However, the foundation stiffness significantly affected the bolt stresses and global frog deformations. A softer foundation with lower stiffness resulted in much larger maximum deflection of the frog, resulting in larger deformations in the bolts and higher shear and bending stresses. These results indicate that a stiffer ballast material or the use of concrete ties or foundation pads to support the diamond could help to mitigate joint deformations and stresses. However, increasing the stiffness of the support system beyond a certain threshold (4.5 ksi in this study) results in diminishing performance returns.

6.2 Recommendations for Future Research Investigating Diamond Crossings

The literature review of prior published research in special track work showed that there is little documentation of behavior for diamond crossings. Although the present study provides valuable new insight, elastic static analysis using amplified wheel loads is not sufficient to completely characterize diamond crossing behavior under the complex dynamic load situation at the flangeway gap. It is recommended that higher fidelity dynamic analysis of diamond crossings be performed in the future. In order to determine the locations of stress concentrations
in actual diamonds, full three-dimensional, solid element models need to be created and subjected to nonlinear dynamic analysis. The proposed research would be of considerably larger scope than that of this report, and could consider modeling the response of flange bearing frogs, solid manganese castings, and reversible casting designs. Additionally, more sophisticated modeling of the subgrade foundation materials and modeling of cross-ties and concrete ties would be advantageous.

Field monitoring is another critical research area that should be pursued in order to better understand behavior and to calibrate computer models. Ideally, it would be very informative to monitor a number of different diamond crossings in the field to determine displacements and state of stress under load. Due to the uncertainty existing around the amount of longitudinal load transferred to the diamond, it is highly desirable to use strain gages to determine the state of stress around the bolted connections during summer and winter months for both new and existing diamonds. This would provide further insight on the exact mechanisms that cause failure of bolted connections in diamond crossings. Although field observation is critical, targeted laboratory studies may also prove valuable for studying aspects of diamond crossing connection behavior. Together, laboratory studies, field data and numerical results from more detailed models will provide a comprehensive new view of diamond crossing behavior and will lead to innovations that improve connection performance.


