ANALYSIS OF THE PERFORMANCE AND FAILURE OF RAILROAD CONCRETE CROSSTIES WITH VARIOUS TRACK SUPPORT CONDITIONS

BY

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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, 2016

Urbana, Illinois

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ABSTRACT

In North America, the use of concrete crossties has increased steadily over the past decade as they have emerged as an economic alternative to timber crossties to accommodate heavy axle freight train loads. As the number of concrete crossties has grown, the importance of understanding the performance of these components has also increased. This is especially true given derailments have been linked to the condition of concrete crossties and therefore an improved understanding is critical to ensure a safe and reliable operation of the track. Currently, the behavior of poorly supported or degraded concrete crossties and their fastening system components is not fully understood, but it is widely accepted that these conditions may have a significant influence on the demands placed on concrete crossties. To quantitatively describe the correlation between support conditions and concrete crosstie performance, laboratory experiments were conducted. The main variables analyzed are bending moments and the gage widening effect due to bending of concrete crossties. In addition to support conditions, the effect of crosstie center cracking is also quantified, initiating a discussion on how to define concrete crosstie failure. Using statistical tools, the experimental results are presented and discussed in this thesis. The findings of this work can impact different groups related to the railway industry, including manufacturers of concrete crossties, railroads, AREMA, the FRA, and research institutions.

"For from him and through him and for him are all things."

Romans 11:36

ACKNOWLEDGEMENTS

I would like to thank CAPES, a Foundation within the Ministry of Education of Brazil, for sponsoring my graduate studies. I am also grateful to the Federal Railroad Administration (FRA) for funding the research project that encompasses the work of this thesis, as well as to the University of Illinois at Urbana-Champaign's Rail Transportation and Engineering Center (RailTEC) for providing all the necessary academic support and laboratory facilities.

It would not be possible to conclude this work without the help of all the faculty, students, and staff in RailTEC. The exceptional support, encouragement, and direction provided by Riley Edwards and Marcus Dersch is gratefully acknowledged. Thanks are due to Prof. Andrawes for the advisory role and to Yu Qian and Matthew Csenge for expert guidance. Indispensable help was provided by additional students, including Phanuwat Kaewpanya, Thomas Roadcap, Po-Yin Liao, Max Silva, Samuel Trapp, Patrick Sullivan, Brevel Holder, Zachary Jenkins, Quinn Todzo, and Zhipeng Zhang. Thank you to all the graduate research assistants that shared the workplace with me, making it a pleasant place to be, especially Henry Wolf, Zhengboyang Gao, Donovan Holder, Kaijun Zhu, Aaron Cook, Xiao Lin, Matthew Greve, Alejandro Alvarez, Arkaprabha Ghosh, Arthur Lima. Moreover, the statistical consulting of Samantha Chadwick and the laboratory assistance of Tim Prunkard and the CEE Machine Shop are thankfully acknowledged.

The industry support from various companies, including UP, BNSF, AMTRAK, Progress Rail Services, GIC, Hanson, LB Foster, TTX, is greatly appreciated. Special gratitude is also expressed to Cameron Stuart (FRA) for coordinating this project and to Hailing Yu (John A. Volpe National Transportation Center) for contributing to its supervision.

In addition, I lack of words to thank all my family for their extensive support and unconditional love in my life. Finally, greatest thanks to Mariane, who joyfully sacrificed personal time, professional aspirations, and family life, with faith in a greater cause and future hope.

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CHAPTER 1: INTRODUCTION

1.1 Introduction

While the scientific understanding of railroad concrete crossties has advanced substantially over the past few years, most research initiatives focus on the performance of healthy track components and ideal track support conditions, leaving deteriorated and suboptimal conditions with little consideration. However, revenue service operation is rarely optimal and track infrastructure problems can compromise the reliability and safety of railroad operations. With the support of the Federal Railroad Administration (FRA), this thesis is part of a larger effort within the Transportation and Engineering Center (RailTEC) at the University of Illinois at Urbana-Champaign (UIUC) that aims to investigate such critical conditions and their consequences on concrete crosstie track.

1.2 Objectives

The objectives of this study are:

- To better understand the critical conditions that increase the risk of accidents on concrete crosstie track in the United States;
- To quantify the effect of various support conditions on the flexural behavior of concrete crossties through laboratory experimentation;
- To initiate a discussion on the effect of concrete crosstie cracking as well as how to define concrete crosstie failure.

1.3 Thesis Outline

The content of this thesis consists of a total of six chapters including this introduction. The scope of each chapter is briefly described in the following paragraphs.

Chapter 2 discusses the critical railroad track conditions with poorly supported or worn concrete crossties and fastening systems. This chapter presents an extensive investigation into the topic by reviewing relevant literature, analyzing the FRA accident database, and reporting the results of a railway industry survey.

Chapter 3 conveys a detailed description of the statistical processes used throughout this thesis. In addition to highlighting the importance of statistics in data processing and encouraging other railroad track infrastructure researchers to incorporate more statistics in their work, the chapter presents useful information for readers that are not familiar with the methods employed.

Chapter 4 presents laboratory experimental results that are useful in associating the flexural demand of concrete crossties with a variety of common support conditions. Since bending moments are one factor that is used in the design of the concrete crosstie, they are quantified for cracked and uncracked concrete crossties. This chapter attempts to investigate how different support conditions and crosstie cracks affect crossties bending moments.

Chapter 5 presents laboratory experimental results that are useful in associating displacements of concrete crossties with a variety of common support conditions. Instead of bending moments, this chapter focuses on increase in railroad track gage due to bending of cracked and un-cracked concrete crossties. Similarly, this chapter explores how different support conditions and cracked crossties can contribute to gage widening, which is identified in this research as a critical problem associated with concrete crossties.

Chapter 6 concludes the thesis by summarizing the main findings, providing recommendations, and discussing potential future work.

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CHAPTER 2: DETERMINATION OF CRITICAL RAILROAD TRACK CONDITIONS WITH POORLY SUPPORTED OR WORN CONCRETE CROSSTIES AND FASTENING SYSTEMS

2.1 Introduction

In North America, the use of concrete crossties has increased steadily over the past decade as they have emerged as an economic alternative to timber crossties to accommodate heavy axle freight train loads (Lutch et al., 2009). As the number of concrete crossties has grown, the importance of understanding the performance of these components has also increased. This is especially true given derailments have been linked to the condition of concrete crossties and therefore an improved understanding is critical to ensure a safe and reliable operation of the track infrastructure (National Transportation Safety Board, 2006; Marquis et al., 2014; Yu et al., 2015). Currently, the behavior of poorly supported or degraded concrete crossties and fastening systems (and their components) is not fully understood, but it is widely accepted that these conditions may have a significant influence on the demands placed on the concrete crosstie and fastening system (Yu et al., 2015). To better understand which conditions influence the risk of accidents on concrete crosstie track, this chapter aims to gather pertinent information to design and execute relevant laboratory experimentation intended to investigate these questions. Sources of data and guidance include the Federal Railroad Administration (FRA) accident database, published literature, the results of a railway industry survey, and extensive input from concrete crosstie experts in the United States rail industry.

2.2 Background

The FRA divides accidents into five groups based on their cause: Infrastructure, Equipment, Signal and Communication, Human Factors, and Miscellaneous (Federal Railroad Administration, 2011). Based on data from the FRA's Office of Safety, the two most common causes of accidents are Miscellaneous and Infrastructure (Table 2.1). As the name suggests, the Miscellaneous category is very broad, including environmental conditions, highway-rail grade crossing accidents, unusual operational situations, etc.

When the Miscellaneous group is removed from the list of categories, infrastructure causes are the most prevalent.

Accident Cause Category	Number of Accidents	Percentage (%)
Infrastructure	3,104	25.8
Equipment	2,510	20.9
Human Factor	2,383	19.8
Signal and Communication	45	0.4
Miscellaneous	3,992	33.2
Total	12,034	100

Table 2.1 Train accidents on Class I mainlines grouped by accident causes

Data obtained from FRA's Office of Safety Analaysis Web Site from 2004 to 2013.

Having identified the primary causes of accidents, there is still the need to narrow down the type of track for a more precise analysis that is relevant to concrete crossties and fastening systems. However, most of the published literature that identifies critical railroad track conditions in North America does not distinguish between the crosstie material (e.g. concrete, timber, etc.) that was present at the location of the accident. Prior research has shown that broken rail and track geometry deviations are the most frequent derailment causes in the United States, and that both conditions lead to higher than average number of cars derailed per accident, a proxy for accident severity (Dick, 2001). While this information is more representative of railway lines with timber crosstie track. Understanding that the primary differences between these two types of track are the crosstie material and the fastening systems typically used with each crosstie type, this literature review focuses on identifying the most critical problems associated with concrete crossties.

2.2.1 Functions and Design of Concrete Crossties

In order to determine the most critical problems of concrete crossties, it is necessary to understand their functions within the railroad track. Any condition that results in crossties being unable to achieve these

purposes will be considered a defective condition. According to Zeman (2010), the roles of railroad crossties are:

- Supporting the rails under load;
- Distributing the stresses at the rail seat to acceptable levels for the ballast layer;
- Maintaining proper geometry of the track structure.

Maintaining proper geometry of the track structure is not an exclusive role of crossties, but it is also shared with other track elements and components (Zeman, 2010). Nevertheless, even though crossties are important to prevent lateral and vertical movements of the track, perhaps their most relevant contribution to maintaining track geometry is to hold the track gage with the assistance of rail fastening systems.

Since the actual fulfillment of the concrete crossties purposes is closely related to their structural design process, it is pertinent to understand the common practices of concrete crosstie design. In addition, even though concrete crossties can be monoblock or twin-block, the latter are out of the scope of this study as they are not commonly used in North America (Wolf, 2015).

There are two prominent methods of designing concrete crossties: the maximum allowable stress approach and the limit states approach. The allowable stress method "ensures that all stresses within the crosstie do not exceed predetermined values" (Murray, 2015), which could lead to an uneconomical scenario by over-designing crossties (Leong, 2007). The limit states method focuses on finding an economically optimal design using probabilistic concepts to guarantee that the crosstie will perform its functions for a given period of time. In North America, the American Railway Engineering and Maintenance-of-Way Association (AREMA) recommends that concrete crossties should be structurally designed based on a conventional maximum allowable stress approach. The AREMA methodology can be summarized in the following steps (American Railway Engineering and Maintenance-of-Way Association, 2014):

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- 1. The rail seat load is obtained as a function of the wheel load, a dynamic impact factor, and crosstie spacing;
- The unfactored rail seat positive bending moment is estimated based on crosstie spacing and crosstie length;
- The maximum allowable rail seat positive bending moment is calculated by factoring the previously calculated rail seat positive bending moment by a speed factor and an annual tonnage factor;
- 4. The maximum allowable rail seat negative, center positive, and center negative bending moments are calculated based on the maximum allowable rail seat positive bending moment and crosstie length.

A more detailed discussion about the assumptions behind the flexural analysis of this process and other common worldwide design practices is presented by Wolf (2015).

Similarly, the limit states design methods also require that the design resistance must be greater than the "effect of design loads" (Ellingwood and Galambos, 1982). The main difference, however, is that this approach avoids factoring the loads, which tend to be the maximum probable loads to occur in a given time period (Ellingwood and Galambos, 1982). For example, if a crosstie is designed with 95% expectancy of lasting 50 years, then the design loads should be those that occur once every 1,000 years. Therefore, in order to make such predictions, it is necessary to have statistical distribution curves providing the magnitude of impact loads imparted by wheel and track irregularities for a particular railway line (Leong, 2007). In addition, there can be many simultaneous limit states, such as serviceability limits of tolerable deformations, acceptable cracking, and maximum vibration. For instance, Murray recommends four limit state categories for concrete crossties, namely: strength, operations, serviceability, and fatigue (Murray, 2015). When developing limit states for concrete crossties, Leong provides one of the most complete lists of defective conditions that would cause these components to fail (Leong, 2007). A modified version of this list is shown below:

- Bottom abrasion that allows for excessive gage widening due to bending deformation;
- Rail seat deterioration (RSD) that allows for excessive gage widening due to rail rotation;
- Cracking that allows for the movement of the fastening systems (e.g. severe rail seat or longitudinal cracking);
- Cracking that allows for excessive gage widening due to bending deformation (e.g. severe center or longitudinal cracking);
- Chemical degradation (e.g. alkali silica reactivity).

The next sections discuss in greater detail two of these defective conditions, namely: "Rail Seat Deterioration", and "Crosstie Cracking".

2.2.2 Rail Seat Deterioration (RSD)

In 2011, researchers at the University of Illinois at Urbana-Champaign (UIUC) conducted an international concrete crosstie and fastening system survey (Van Dyk et al., 2012). They presented a rank of typical failure modes of concrete crossties and fastening system both internationally and domestically. Table 2.2 summarizes the findings of this study, with common crosstie and fastening system problems ranked from 1 to 8, with 8 being most critical. UIUC researchers found that rail seat deterioration (RSD) was considered the most critical failure associated with concrete crossties in the U.S. Zeman et al. (2010) identified five mechanisms that can cause RSD: abrasion, crushing, freeze-thaw cracking, hydraulic-pressure cracking, and hydro-abrasive erosion. Kernes (2014) investigated the mechanics of abrasion on concrete crosstie rail seats, and Greve et al. (2015) examined the effect of rail seat load distribution on RSD.

Failure	Rank
Deteroriation of concrete material beneath the rail	6.43
Shoulder/fastening system wear or fatigue	6.38
Cracking from dynamic loads	4.83
Derailment damage	4.57
Cracking from center binding	4.50
Tamping damage	4.14
Other (e.g., manufactured defect)	3.57
Cracking from environmental or chemical degradation	3.50

Table 2.2 Ranked list of critical concrete crosstie and fastening system problems in the U.S. (higher numbers indicate increased criticality) (Van Dyk et al., 2012)

Two Amtrak derailments caused by RSD in 2005 and 2006 further emphasized the importance of the topic (National Transportation Safety Board, 2006; Marquis et al., 2011), and the potential for RSD to lead to a derailment. After these accidents, the FRA formed a task force to evaluate the problem, which resulted in changes to the Code of Federal Regulations (CFR) 213, the FRA Track Safety Standards (TSS), introducing new regulations relating to concrete crossties (Clouse, 2012). Currently, the FRA TSS state that concrete crossties should not be deteriorated or abraded at any point under the rail seat to a depth of ½ inch (12.7 mm) or more (Federal Railroad Administration, 2015b). Additional research relating RSD to rail rollover and gage widening was published by researchers from the John A. Volpe National Transportation Systems Center (Volpe) (Choros et al., 2007; Marquis et al., 2011).

2.2.3 Crosstie Cracking

A different point of view is presented by Taherinezhad et al. (2013), who suggest that cracking could be the most common failure of prestressed concrete crossties worldwide. They discuss the fact that the use of high strength concrete increases the crosstie brittleness when compared to normal concrete strength, making it more prone to cracking (Taherinezhad et al., 2013). However, categorizing the types of cracks is helpful since there are variations of how they manifest themselves and the potential risk (or lack thereof) they pose. Disregarding chemical and freeze-thaw cracks, there are three common types of concrete crosstie cracking: center, rail seat, and longitudinal (Clouse, 2012).

Center cracks are located at the crosstie midspan. They typically start on the top of the crosstie and grow vertically in the direction of the bottom, resulting from high center negative bending moments imposed by improper crosstie support under the rail seats. A CSX derailment on Metro-North tracks in the Bronx, NY, in 2013 is an example of center bound concrete crossties being a critical factor contributing to the accident (Marquis et al., 2014). Using a finite element (FE) model of a prestressed concrete crosstie, other researchers concluded that gaps between the concrete crosstie and ballast at the rail seat region considerably increase the flexural demand at the crosstie center (Chen et al., 2014). For the crosstie type considered in the study, a gap larger than 0.1 in (2.54 mm) resulted in tensile cracking of concrete at the top surface of crosstie midspan. However, predicting the crosstie support conditions to determine the chance of center cracking is nontrivial. Experiments conducted at TTC revealed significant variability in pressure distribution under concrete crossties, even within the same type of track and between adjacent crossties (McHenry, 2013).

As the name suggests, rail seat cracks are located under the rail bearing area of the crosstie. In most cases, the crack initiates at the bottom or sides of the crosstie and propagates vertically to its top (Clouse, 2012). Commonly, rail seat cracking is the result of a combination of stiff track and high impact loads (Kaewunruen and Remennikov, 2010). Using a FE model, researchers showed that rail seat cracks can occur when the crosstie is supported by a uniform and homogeneous ballast layer (Yu et al., 2011). However, even though published literature indicates it is a frequent deteriorated condition (ZETA-TECH, 2010), focused conversations with railway industry experts showed that rail seat cracking is not a common concrete crosstie problem in North America. While there is not sufficient data to determine the exact occurrence or severity of rail seat cracks, some researchers suggest that rail seat cracks might have more severe consequences for the crosstie deflections and stresses than center cracks (Domingo et al., 2014).

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The third type of cracks are longitudinal, which, according to the FRA, are "horizontal through the crosstie and extend parallel to its length" (Clouse, 2012). Commonly associated with high stresses in the vicinity of prestressing wires and the indented wire geometry (Rezaie et al., 2012; Mayville et al., 2014), longitudinal cracks pose a challenge for concrete crosstie manufacturers who consider increasing presstressing forces to improve flexural capacity (Harris et al., 2011). The FRA report on performance of concrete crossties on Amtrak's Northeast Corridor (NEC) indicates that longitudinal cracking was the predominant cracking mode associated with the replacement of crossties on the NEC (Mayville et al., 2014).

Improper support conditions, high impact wheel loads, and high stresses at the prestressing wires are not the only causes for concrete crosstie cracking. Chemical reactions (e.g. ASR and delayed ettringite formation (DEF)), manufacturing defects, improper design and in service vibration, and other causes can also result in concrete crosstie cracking. Chemical reactions have been critical causes of crosstie cracking in Sweden, where approximately 500,000 concrete crossties were identified as showing signs of DEF (Thun et al., 2008). Similarly, ASR contributed to the longitudinal cracking of crossties installed in the 1990's on Amtrak's Northeast Corridor (NEC) (Mayville et al., 2014). ASR also caused the failure of more than 350,000 crossties installed in the 1970's by Canadian National (ZETA-TECH, 2010). Similarly, vibration can be an adverse factor for concrete crossties, causing most damage at resonant frequencies of the crosstie's first five modes of vibration (Kaewunruen and Remennikov, 2006). These frequencies commonly include corrugation-passing frequencies (Grassie and Cox, 1984), meaning that dynamic damage of trains moving on corrugated rail can be worse than it would otherwise be in the presence of other track irregularities.

2.3 Analysis of Data from FRA Accident Database

One way to determine critical railroad track conditions, in terms of causing accidents, is to analyze data contained within the FRA accident database. While the specified accident causes depend on the

thoroughness and accuracy of each accident's investigation and subsequent documentation, these data are provided by individual railroads and are capable of indicating general trends in the U.S. Using the FRA database, an analysis of the accidents caused by infrastructure problems on main lines of Class I railroads from 2004 to 2013 was performed. Six FRA accident cause codes were considered relevant for this work (Table 2.3).

Code	Description
T110	Wide gage (due to defective or missing crossties)
T111	Wide gage (due to defective or missing spikes or other rail fasteners)
T205	Defective or missing crossties (not resulting in wide gage)
T206	Defective spikes or missing spikes or other rail fasteners (not resulting in wide gage)
T001	Roadbed settled or soft
T105	Insufficient ballast section

Table 2.3 FRA accident codes used in this study

First, the average number of cars derailed per accident for each of the six selected codes was computed and compared to the average number of cars derailed per accident for all infrastructure-related codes (Figure 2.1). The average number of cars derailed serves as a proxy for accident severity. Second, the total number of accidents of each accident code was compared to the average incidence of all infrastructure accident codes from 2003 to 2014 (Figure 2.1). These data were plotted on a frequency versus severity graph (Figure 2.1). The accident causes that were above average for both frequency and severity (i.e. in the upper right quadrant of the graph) were considered critical and would be further investigated through our experimental study (Figure 2.1).



Figure 2.1 Track-caused derailment analysis for all types of track

It is important to note that, for an accident or incident to be reportable to the FRA, it has to be classified in one of three primary groups: "Highway-Rail Grade Crossing", "Rail Equipment", or "Death, Injury and Occupational Illness" (Federal Railroad Administration, 2015a). However, this analysis only accounts for accidents that had material damage above the FRA official reportable threshold (\$9,900 for year 2013), which includes all rail equipment related accidents and part of grade crossing related incidents (Federal Railroad Administration, 2015a). With this information, one can conclude that the overall average number of cars derailed is not underestimated due to the influence of accidents with material damage lower than the reportable threshold.

However, this analysis does not differentiate between crosstie materials since the FRA database has no field that relates to crosstie material, design, or vintage. In order to understand the critical infrastructure related accidents on concrete crosstie track, the type of crosstie had to be identified to determine the crosstie material for every accident location where one of the six accident codes from Table 2.3 was named. Therefore, an investigation was conducted, which consisted of analyzing aerial imagery of the accident sites in order to identify the crosstie type. Since the available images were of 2014, it is possible that they were not exact representations of the crosstie material at the time the accident occurred. However, this error was assumed to be negligible, as it is not likely that many sites would have experienced changes in crosstie type in less than 10 years. As a result, a new frequency versus severity plot was developed specifically for concrete crosstie track using only the six accident codes from Table 2.3 (Figure 2. 2). Additionally, the average number of cars derailed and average incidence in ten years are based on the six studied accident codes, not all infrastructure-related accidents.



Figure 2.2 Track-caused derailment analysis for concrete crosstie track

When considering all types of track, the problems that rank highest in terms of both severity and frequency are "wide gage due to defective fastening system" (T111), "wide gage due to defective crossties" (T110), and "roadbed settled or soft" (T001). The results are similar for concrete crosstie track, except that "wide gage due to defective fastening systems" has a lower frequency compared to the other cause codes. This difference could indicate that elastic fasteners, which are required for concrete crossties, are more effective in restraining gage than cut spikes in timber crossties.

In addition, the high criticality of "roadbed settled or soft" in both cases is a sign that poor track support conditions are a critical element that should be further explored. In concrete crossties, the effect of poor support conditions can manifest themselves through cracking. Cracking could happen, for example, under center binding or in the presence of substructure with high stiffness (Clouse, 2012). However, explaining the support conditions of crossties is non-trivial and there is need for additional research in this area.

Derailments on concrete crosstie track due to "wide gage due to defective crossties" occurred at a higher than expected frequency, given concrete crossties are known for holding gage well. However, most of the degraded conditions of concrete crossties can contribute to gage widening. For example, RSD can generate rail cant deficiency, which can allow the rail to roll and alter gage (Choros et al., 2007). Similarly, crossties with reduced flexural capacity from cracking or abrasion of their bottom surface might suffer significant deflections under loading, which also adds to the loaded gage. Therefore, it is likely that wide gage due to defective concrete crossties is a critical problem in the U.S. because multiple factors can contribute to its occurrence.

2.4 Survey of Railway Industry Experts

A survey relating to worn and degraded conditions in concrete crosstie track was developed to identify critical problems associated with concrete crosstie track in North America. In contrast to the broader survey previously cited in this chapter (Van Dyk et al., 2012), this survey was shorter and more specific,

with only four questions. Similarly, however, this survey was distributed to experts at railroads, crosstie suppliers, and industry and academic research institutions. Topics included the criticality of possible track defects, qualitative assessment of FRA accident codes, the identification of combinations of deteriorated track conditions that can lead to derailments, and noting potential areas of track infrastructure laboratory experimentation and research. Fourteen individuals took the survey, representing railroads, concrete crosstie manufacturers, and research institutions.

2.4.1 Survey Findings

When asked about the criticality of various conditions with respect to the occurance of accidents on concrete crosstie tracks, RSD emerged as the most critical concrete crosstie condition (Table 2.4). It should be noted, however, that the first six items are directly related to the condition of concrete crossties and fastening systems, while substructure problems are listed with relatively low criticality.

Failure	Rank
Rail seat deterioration and other forms of rail cant deficiency	4.57
Worn or missing shoulder	4.14
Worn or missing insulator	3.79
Missing clip	3.71
Center negative crosstie bending	3.43
Missing rail pad	3.36
Fouled ballast	3.21
Insufficient depth of ballast	3.00
Weak subgrade	3.00
Concrete crosstie with deteriorated bottom	2.93
Rail seat positive crosstie bending	2.43

Table 2.4 Criticality of track structure conditions; ranked from 1 to 5, with 5 being the most critical

A similar question was posed regarding the specific FRA accident codes referenced in this chapter, and the respective responses are represented in Table 2.5. In addition, as reported on Table 2.5, the survey also indicates that "wide gage due to defective or missing crossties" is the top ranked accident

code reported to the FRA, confirming the analysis of the FRA database. However, the accident code "roadbed settled or soft" was ranked quite low, which does not align with the findings from the FRA database analysis (Figure 2. 2). This inconsistency may be an indication that "roadbed settled or soft" is perhaps reported to the FRA as an accident cause more often than it should be, or the fact that this term might have been undervalued by survey respondents as it is reflective of a broad set of track conditions.

Table 2.5 Criticality of FRA accident codes for concrete crosstie track; ranked from 1 to 5, with 5 being the most critical

Description	Rank
Wide gage (due to defective or missing crossties)	4.33
Wide gage (due to defective or missing spikes or other rail fasteners)	4.25
Defective or missing crossties (not resulting in wide gage)	3.64
Defective spikes or missing spikes or other rail fasteners (not resulting in wide gage)	3.42
Roadbed settled or soft	3.25
Insufficient ballast section	3.00
	Description Wide gage (due to defective or missing crossties) Wide gage (due to defective or missing spikes or other rail fasteners) Defective or missing crossties (not resulting in wide gage) Defective spikes or missing spikes or other rail fasteners (not resulting in wide gage) Roadbed settled or soft Insufficient ballast section

Table 2.6 indicates pairs of track conditions that the respondents felt could lead to a derailment, Table 2.6 shows that eleven people responded that the combination of "worn or missing shoulder" and "worn or missing insulator" could lead to a derailment. The same is true for the pair "worn or missing shoulder" and "RSD and other forms of rail cant deficiency". Second to these, the combination of "worn or missing shoulder" with "missing clip" and the pair "center negative crosstie bending" and "concrete crosstie with deteriorated bottom" were considered as potential derailment causes by ten respondents. A common factor that these four pairs have in common is the type of derailment they could potentially lead to: wheel drop due to gage widening. Table 2.6 Pairs of problems that could lead to a derailment; each number indicates the quantity of votes received by the pair represented in that cell (out of 14 responses)

Concere crossile with deterio aled boild	A LAND AND AND AND AND AND AND AND AND AND	A HEAL THE	all set of the should be	Certer interaction	and closelicience	40000000000000000000000000000000000000	Santice Long	27 000 th 00 100	Weat balls		8
Concrete crosstie with deteriorated bottom		3	3	2	2	8	5	10	5	7	8
Missing rail pad			7	8	5	1	1	2	2	2	4
Worn or missing insulator				11	8	1	0	2	3	1	3
Worn or missing shoulder					10	11	3	2	2	3	4
Missing clip						9	1	4	2	0	4
RSD and other forms of rail cant deficiency							3	4	4	3	7
Rail seat positive crosstie bending								1	2	4	5
Center negative crosstie bending									7	7	7
Fouled ballast										8	8
Insufficient depth of ballast											7
Weak subgrade											

Finally, Table 2.7 summarizes the most common responses relating to what degraded conditions of track structure the respondents would like to see tested in a laboratory. "Cracked crossties" emerged as the most requested topic for laboratory experimentation on concrete crosstie with four references. This is especially significant considering that this was an essay question where the respondents could freely recommend degraded conditions of concrete crosstie track. It is also worthy to note that "center negative crosstie bending" can be considered a specific subtopic of "crosstie support condition", in which case their total number of votes would result in it being the most common response.

Торіс	Votes
Cracked crossties	4
Crosstie support condition	3
Saturated ballast (wet ballast)	3
Center negative crosstie bending	3
Worn fastening systems	3

Table 2.7 Most recommended topics for laboratory tests (out of 14 responses)

2.5 Conclusions

This chapter focused on identifying critical infrastructure conditions for concrete crosstie track with the objective of informing future laboratory experimentation. In addition to the literature review, an analysis of ten years of data from the FRA accident database and a railway industry survey was conducted. Published literature shows that RSD and concrete crosstie cracking are the most common failures that were noted for concrete crossties. However, rail seat cracking appears to be less frequent than longitudinal and center cracking in the U.S. Based on the analysis of FRA accident data, it was concluded that the accident codes "wide gage due to defective crossties" and "roadbed settled or soft" are both more frequent and more severe than average for concrete crosstie track.

The survey responses indicated that railway industry experts consider "wide gage due to defective crossties" to be more critical than "roadbed settled or soft". The responses also confirmed the high criticality of RSD as a problem in concrete crosstie track. Moreover, the survey results supported the idea that wheel drop due to gage widening might be the leading type of derailment due to combined infrastructure problems of concrete crosstie track. One additional finding from the survey was that crosstie support condition (including center binding) and cracked crossties were the most recurring topics recommended for laboratory experimentation.

Therefore, it seems that most problems of concrete crossties are not related with their functions of supporting the rails or transmitting the loads to the ballast, but with its role of restraining track gage. RSD, concrete crosstie center cracking, and crosstie bottom deterioration are all issues that can contribute to gage widening. As such, there is a need to better quantify the acceptable service limits of these conditions that would not disturb railway operations nor compromise safety. However, in most cases these limits can be a function of the crosstie support conditions, the fastening system conditions, and even the crosstie design.

As noted by other researchers, the root causes for problems in concrete crosstie track are not always clear (Yu et al., 2015). The broad list of degraded conditions in concrete crosstie track indicates the complexity of the railroad track as a whole and attempting to narrow down the critical conditions could lead to an omission of important terms. To reduce the frequency of concrete crosstie track problems, further studies are necessary not only to better understand each individual topic, but also to analyze the track as a system.

CHAPTER 3: APPLICATION OF BASIC STATISTICAL CONCEPTS FOR CONCRETE CROSSTIE RESEARCH

3.1 Introduction

The use of statistics in data analysis is extremely powerful as it helps transform raw data into information by using standard and reliable procedures. However, not every investigator is familiar with the statistical tools used by prior researchers that are referenced in published material, making it difficult for the reader to follow the data processing steps. Because it is important that academic publications present all information necessary to replicate the work at some point in the future, and to encourage the use of statistics by other researchers in the field of railroad track infrastructure engineering, this chapter conveys a detailed description of the statistical processes used in this thesis. The chapter is structured to first provide some fundamental statistics concepts, and later to introduce the analysis of variance and the Fisher's least significant difference procedures. Most of this content is derived from the book *An Introduction to Statistical Methods and Data Analysis*, by Ott and Longnecker (2008) and it is not intended to be an exhaustive explanation of the topics referenced.

3.2 Fundamental Concepts

3.2.1 Population and Sample

In most cases, statistical conclusions are based on the data from a *sample*, which is "any subset of measurements from the population" (Ott and Longnecker, 2008). The *population*, however, is the "set of all measurements of interest to the sample collector" (Ott and Longnecker, 2008). In addition, a measurable characteristic of a population is called a *parameter*, while a characteristic of a sample is called a *statistic*. For example, railroad track engineers could be interested in knowing the mean lifetime of concrete crossties in the United States (U.S.), but it is not likely that they would be able to find the lifetime of every single concrete crosstie in the entire country. Therefore, the engineers would have to

use the mean lifetime of certain, selected concrete crossties to make inferences about the complete set of concrete crossties in the U.S. In this illustration, the population is represented by all the lifetimes of concrete crossties in the U.S., and the parameter is the population mean. Correspondingly, the sample is the selected subset of lifetimes of concrete crossties in the U.S., and the parameter crossties in the U.S., and the statistic is the sample mean.

3.2.2 The Box Plot

Once a sample dataset is provided to be analyzed, it is common to utilize various graphical representations to visualize the data, one of which is the *box plot*. Box plots are a valuable resource for comparing distributions between different datasets while keeping the possibility of checking the symmetry of the distributions and the measures of central tendency (mean and median) that a histogram would provide. Box plots also provide an easy way to visualize outliers. The elements of the box plots used in this research are:

- Q_1 : The 25th percentile (lower quartile);
- Q_2 : The 50th percentile (median);
- Q_3 : The 75th percentile (upper quartile);
- *IQR*: The difference between Q₃ and Q₁ (inter quartile range);
- The mean of the dataset;
- Whiskers extending to the largest and smallest values which are within the inner fences;
- Moderate outliers: values outside the inner fences but still within the outer fences (if any);
- Extreme outliers: data points outside the outer fences (if any).

Usually, the fences of box plots are not shown in the figures, but they are theoretical values calculated as follow:

- Upper outer fence: $Q_3 + (3 \times IQR)$;
- Upper inner fence: $Q_3 + (1.5 \times IQR)$;
- Lower inner fence: $Q_1 (1.5 \times IQR)$;

• Lower outer fence: $Q_1 - (3 \times IQR)$.

Figure 3.1 illustrates the main components of a box plot. Graphically, asterisks would represent extreme outliers (as opposed to circles for moderate outliers).



Figure 3.1 Elements of a box plot

3.2.3 Central Limit Theorem and Sample Size

As previously mentioned, in most cases the parameter is unknown and the research conclusions must be based on some statistic. However, while the parameter has only one possible value, the statistic can vary based on each sample. Suppose track engineers wanted to know what the mean wheel load is on a particular site of a railroad line for a given month. The mean wheel load measured in one day may differ from the mean wheel load measured on the next day for that same site – these are sample means. Yet, the population mean, which is the mean wheel load in that site for the entire month, is a constant number.



Figure 3.2 Sample means can differ from each other and from the population mean

When the parameter of interest cannot be found, it is necessary to account for the variability of a statistic by considering its sampling distribution. This is a fundamental step to determine the minimum sample size for an experiment. For the particular case when the parameter of interest is the population mean, the *Central Limit Theorem* is extremely useful. Ott and Longnecker (2008) state this theorem and its conclusions as follows:

"Let \overline{y} denote the sample mean computed from a random sample of n measurements from a population having a mean μ , and finite standard deviation σ . Let $\mu_{\overline{y}}$ and $\sigma_{\overline{y}}$ denote the mean and standard deviation of the sampling distribution of \overline{y} , respectively. Based on repeated random samples of size n from the population, we can conclude the following:

- 1. $\mu_{\overline{y}} = \mu$
- 2. $\sigma_{\bar{y}} = \sigma / \sqrt{n}$
- 3. When n is large, the sampling distribution of \overline{y} will be approximately normal (with the approximation becoming more precise as n increases).
- 4. When the population distribution is normal, the sampling distribution of \overline{y} is exactly normal for any sample size n." (Ott and Longnecker, 2008).

Therefore, the sample size *n* can be derived from the second conclusion of the Central Limit Theorem. However, $\sigma_{\bar{y}}$ (the standard deviation of the sampling distribution of \bar{y}) is usually unknown. Nevertheless, this problem can be overcome if the population of \overline{y} is normal, as, using standard scores (z-values), *n* can be written as:

$$n = \frac{(z_{\alpha/2})^2 \sigma^2}{D^2}$$
(3.1)

where *D* is the detectable deviation of the sample mean relative to the population mean and the level of confidence is $100(1 - \alpha)$ %. However, usually the standard deviation σ is not known either. In this case, it can be approximated by a sample standard deviation or by a guess based on expert judgment. Even though it will not be exact, such approximations can estimate the order of magnitude of the needed number of replicates in an experiment. Once the experiments have started the sample variance can be measured and the number of replicates can be recalculated and adjusted.

The commonly called *Empirical Rule* (Ott and Longnecker, 2008) can be of great assistance in this process of estimating σ , as it correlates how much of the data is located within intervals of multiple standard deviations away from the mean, which is illustrated in Figure 3.3. Suppose that the aforementioned track engineers interested in finding the mean lifetime of concrete crossties in a given railway line want to be 95% confident (i.e. α equal to 0.05) that their estimated mean lifetime is accurate within plus or minus 2 years. Based on experience, they could expect, for instance, that about 70% of the crossties would be within plus or minus 10 years away from the mean lifetime, thus leading to a standard deviation of 10 years (Figure 3.3 shows that 68.26% of the data are within $\mu \pm \sigma$ in a normal distribution). Therefore, they can estimate the number of crossties they need to monitor by applying Equation 3.1:

$$n = \frac{(z_{\alpha/2})^2 \sigma^2}{D^2} \Rightarrow n \approx \frac{(z_{0.05/2})^2 10^2}{2^2} \Rightarrow n \approx \frac{(1.96)^2 100}{4} \Rightarrow n \approx 96.04 \Rightarrow n \approx 97$$

which shows that monitoring the lifetime of 97 concrete crossties would be sufficient to meet their requirements assuming the initial assumptions were accurate. An additional way of estimating the population standard deviation is to assess the range of the data and divide it by four (95.44% of the data are within $\mu \pm 2\sigma$ in a normal distribution, as shown in Figure 3.3). Back to the example, suppose the

best concrete crossties last up to 60 years, but the poorest performing ones have a life of only 20 years. This would result in a range of 40 years and, therefore, an approximate standard deviation of 10 years.



Figure 3.3 Normal distribution divided in intervals defined by the mean μ and standard deviation σ

3.3 Analysis of Variance

In general, it is easier to derive a statistical conclusion from experiments that compare two populations than those comparing three or more populations. A simple hypothesis test could be sufficient to determine whether applying epoxy on rail seats of concrete crossties attenuate rail seat deterioration (RSD). In this case, there are only two populations: the measurements of RSD for concrete crossties without epoxy, and for those with epoxy. However, experiments are not always as simple, and there could be different levels epoxy application, such as low, medium, and high amounts of epoxy. This would result in four populations being compared: the measurements of RSD for concrete crossties

without epoxy, and for those with varying amounts of epoxy. For cases where multiple population means are being compared, a simple hypothesis test would not be sufficient. An *analysis of variance* (ANOVA or AOV), however, might be a suitable tool to draw statistical conclusions in such cases.

The ANOVA testing procedures start with the null hypothesis that all the population means are equal, and the alternative hypothesis is that at least one of the population means is different from the others. If the null hypothesis is true, then the sample means should be similar, but not necessarily identical. The variance of these sample means is the main variable of the analysis (thus the name "analysis of variance") and the null hypothesis is rejected or not based on some threshold level of variability. Figure 3.4 illustrates the scenario where hypothetical distributions of three samples have similar means. The question that ANOVA attempts to answer is whether these samples all come from the same population.



Figure 3.4 Illustration of sample distributions with similar means

3.3.1 Completely randomized design

Before describing the ANOVA details, it is important to highlight that all the work of this thesis is based on a *completely randomized design* of experiment (CRD), which, by definition, has no restrictions imposed on randomization. For an experiment with one factor, *i* treatments and *j* replications, the CRD linear model can be written as:

$$y_{ij} = \mu + \tau_i + \varepsilon_{ij} \tag{3.2}$$

where,

 y_{ii} : The jth observed value of the ith treatment.

 μ : Grand mean.

 τ_i : Fixed effect of the ith treatment.

 ε_{ii} : Random error (residual) of the jth observation of the ith treatment.

The example of applying epoxy to the rail seat of concrete crossties to minimize rail seat deterioration can illustrate this model. In that case, the "factor" is the epoxy application and there are four "treatments" (also called "factor levels"): no epoxy, low epoxy, medium epoxy, and high epoxy. The grand mean takes into account all treatments, and the fixed effect is the expected deviation from the grand mean for a particular treatment. However, the expected value of a measurement may not be the real value observed, and this difference is called error or residual (Figure 3.4).



Figure 3.5 Representation of the CRD model described by Equation 3.2

When restrictions are imposed to randomization, other experimental designs should be implemented, such as *Randomized Complete Block Design* (RCBD), *Latin Square Design*, and *Split Plot Design*. However, these and others are beyond the scope of this thesis.

3.3.2 ANOVA Assumptions

To understand the logic of ANOVA, it is also necessary to know all of its assumptions. There are three major assumptions that are made in the ANOVA process:

- Normality: the assumption that all the populations considered in the analysis have a normal distribution. Commonly, this condition is verified with a normality test with the combined residuals of all populations. In this thesis, the Shapiro-Wilk test for normality (Shapiro and Wilk, 1965) is used. However, it has often been reported that violation of the normality assumption should be of little concern when performing an ANOVA, especially for large sample sizes (Glass et al., 1972).
- Homogeneity of Variances: the assumption that all the populations considered in the analysis have equal variance. In this thesis, this condition is formally verified using the Brown and Forsythe's test for homogeneity of variance (Brown and Forsythe, 1974). The homogeneity of variance assumption is crucial for the ANOVA process and a transformation of the data may be necessary if this condition is not met with the original data. Ott and Longnecker (2008) define the transformation of the sample data as "a process in which the measurements on the original scale are systematically converted to a new scale of measurement". A more detailed discussion on data transformation is provided in their book, *An Introduction to Statistical Methods and Data Analysis*.
- <u>Independence</u>: the assumption that the residuals are independent. In simple terms, independent measurements are not correlated to other measurements. For example, there can be a time correlation between measurements if the results are expected to be time dependent. Usually this assumption is not formally verified, but a "careful review of how experiment or study was conducted" is recommended by Ott and Longnecker (2008). Nevertheless, a correlation analysis can be performed using metrics such as the Durbin-Watson statistic (Durbin and Watson, 1950; Durbin and Watson, 1951). In addition, a more detailed study on

variance-covariance matrixes can be useful to determine whether the data are independent or not (Robert O. Kuehl, 1999).

3.3.3 ANOVA Logic and Steps

When two normally distributed populations have the same variance, the ratio between the variances of random samples of these populations will follow an F distribution. The ANOVA is based on an F test where the ratio between two calculated variances is the test statistic, which is commonly called $F_{calculated}$ (Equation 3.3). The denominator of this test statistic is the common variance *within* a sample, also named mean square within (MSW) or mean square error (MSE), while the numerator is the variance *between* the samples, named mean square between (MSB) or mean square treatment (MST). The former is a representative average of the variances of each sample, while the latter is based on the variance of the sample means and the number of replicates (analogous to the second conclusion of the Central Limit Theorem). These two variances are both estimates of the same parameter – the common population variance – and should be similar if the null hypothesis is true by stating that all the populations have the same mean. This would lead $F_{calculated}$ to be close to one. However, if any of the population means are different from the others, the "variance between", which is based on the variance of the sample means, will be affected. Therefore, the more the population means differ from each other, the farther from unity the test statistic becomes and the null hypothesis is more likely to be rejected.

$$F_{calculated,df_1,df_2} = \frac{MSB}{MSW} = \frac{MST}{MSE}$$
(3.3)

where,

 $F_{calculated,df_1,df_2}$: ANOVA test statistic, which is associated with the F distribution. df_1 : Treatment degrees of freedom ($df_1 = t - 1$, where t is the total number of treatments). df_2 : Error degrees of freedom ($df_2 = n_T - t$, where n_T is the total number of measurements). MST: Mean square treatment (between samples). MSE: Mean square error (within samples).

The "mean square" terms are calculated based on the "sums of squares", as defined in Equations 3.4 to 3.6. Once all the terms are calculated, the results are typically reported in a table format, as shown in Table 3.1.

$$MST = \frac{\sum_{i} n_{i} (\bar{y}_{i.} - \bar{y}_{..})^{2}}{t - 1} = \frac{\sum_{i} n_{i} (\hat{\tau}_{i})^{2}}{t - 1} = \frac{SST}{df_{1}}$$
(3.4)

$$MSE = \frac{\sum_{i} \sum_{j} (y_{ij} - \bar{y}_{i.})^{2}}{n_{T} - t} = \frac{\sum_{i} \sum_{j} (\varepsilon_{ij})^{2}}{n_{T} - t} = \frac{SSE}{df_{2}}$$
(3.5)

$$TSS = \sum_{i} \sum_{j} (y_{ij} - \bar{y}_{..})^2 = SST + SSE$$
(3.6)

where,

- SST: Sum of squares treatment.
- *SSE*: Sum of squares error.
- *TSS*: Total sum of squares.
- \bar{y}_i : Mean value observed for the ith treatment.
- $\bar{y}_{\rm u}$: Mean value of all measurements.
- n_i : The number of replicates for the ith treatment.
- n_T : The grand total number of observations.
- $\hat{\tau}_i$: Observed effect of the ith treatment (from sample).

Table 3.1 ANOVA Table and Typical Variables Included

Source	Degres of freedom	Sum of Squares	Mean Square	F value	Pr > F
Treatments	t-1	SST	MST	$F_{calculated}$	p-value from F distribution
Error	$n_T - t$	SSE	MSE		
TOTAL	$n_{T} - 1$	TSS			

To conclude the hypothesis test, the resulting p-value (last column of Table 3.2) is compared to a chosen significance level (i.e. alpha value). If the significance level is greater than the p-value, the null hypothesis that all population means are identical is rejected, and the effects are considered significant.

3.3.4 The Case of Two Factors

Looking again to the RSD example, in addition to epoxy application, there could be a second factor, such as introducing metallic fine aggregates (MFA) in the concrete mix when manufacturing the crosstie specimen. In this case, the previous method would not be sufficient, as only one factor has been considered in the ANOVA so far. Therefore, there is the need to explain the same process with two factors, which is commonly called two-way ANOVA. In the case of two factors, A and B, the CRD linear model would be written as:

$$y_{ijk} = \mu + \tau_i + \gamma_j + (\tau\gamma)_{ij} + \varepsilon_{ijk}$$
(3.7)

where,

 y_{iik} : The kth observed value with the ith level of factor A and the jth level of factor B.

 μ : Grand mean.

- τ_i : Fixed effect of the ith level of factor A.
- γ_i : Fixed effect of the jth level of factor B.
- $(\tau\gamma)_{ij}$: Fixed effect of the interaction of the ith level of factor A with the jth level of factor B.
- ε_{ijk} : Residual of the kth observation with the ith level of factor A and the jth level of factor B.

Equation 3.7 includes a new term, the interaction effect $(\tau \gamma)_{ij}$. This term accounts for differences between simple effects of a factor at different levels of the other factor. For example, suppose that a rail seat with no epoxy or MFA would deteriorate 0.50 inches naturally. Applying a high amount of epoxy would perhaps decrease the deterioration by 0.05 inches, while including MFA in the concrete mix could reduce it by 0.30 inches, as can be inferred from Shurpali et al. (2013). However, is it possible to conclude that using both MFA and high amount of epoxy would reduce RSD by 0.35 inches? Probably
not, as the epoxy layer could have a smaller contribution to reducing RSD in the presence of this different mix of concrete. Therefore, there is an interaction between the effect of applying epoxy and the effect of introducing MFA in the concrete mix.

In the two-way ANOVA, an F value is calculated for each of the main effects and for the interaction factor, thus leading to three p-values and three hypothesis tests. For samples of the same size, the "mean squares" and "sums of squares" are defined in Equations 3.8 to 3.12. Table 3.2 shows how the results are typically reported.

$$MSA = \frac{bn\sum_{i}(\bar{y}_{i..} - \bar{y}_{..})^{2}}{a - 1} = \frac{SSA}{a - 1}$$
(3.8)

$$MSB = \frac{an\sum_{j} (\bar{y}_{.j.} - \bar{y}_{..})^{2}}{b-1} = \frac{SSB}{b-1}$$
(3.9)

$$MSAB = \frac{n\sum_{i}\sum_{j}(\bar{y}_{ij.} - \bar{y}_{i..} - \bar{y}_{.j.} + \bar{y}_{..})^{2}}{(a-1)(b-1)} = \frac{SSAB}{(a-1)(b-1)}$$
(3.10)

$$MSE = \frac{\sum_{i} \sum_{j} \sum_{k} (y_{ijk} - \overline{y}_{ij.})^{2}}{ab(n-1)} = \frac{SSE}{ab(n-1)}$$
(3.11)

$$TSS = \sum_{i} \sum_{j} \sum_{k} (y_{ijk} - \bar{y}_{...})^2 = SSA + SSB + SSAB + SSE$$
(3.12)

where,

MSA: Mean square of factor A.

SSA: Sum of squares of factor A.

MSB: Mean square of factor B.

SSB: Sum of squares of factor B.

MSAB: Mean square of interaction factor AB.

SSAB: Sum of squares of factor AB.

 $\bar{y}_{i..}$: Mean value observed for the ith level of factor A.

 $\bar{y}_{,j}$: Mean value observed for the jth level of factor B.

- $\bar{y}_{...}$: Mean value of all observations.
- *n*: The number of replicates within a sample (these formulae are for sample of the same size).
- *a*: The number of levels of factor A.
- *b*: The number of levels of factor B.

Table 3.2 Two-Way ANOVA Table and Typical Variables Included

Source	Degres of freedom	Sum of Squares	Mean Square	F value	Pr > F
Main Effect A	a – 1	SSA	MSA	MSA/MSE	p-value from F distribution
Main Effect B	b-1	SSB	MSB	MSB/MSE	p-value from F distribution
Interaction AB	(a - 1)(b - 1)	SSAB	MSAB	MSAB/MSE	p-value from F distribution
Error	ab(n-1)	SSE	MSE		
TOTAL	$n_{T} - 1$	TSS			

According to Ott and Longnecker (2008), "the first test of significance must be to test for an interaction between factors A and B, because if the interaction is significant then the main effects may have no interpretation". Therefore, if the interaction between factors is significant, making conclusions about the main effects can be challenging or impossible. However, when the interaction effect is not significant, the analysis of the main effects can be carried out regularly.

3.4 Fisher's Least Significant Difference

The hypothesis test of ANOVA indicates whether all the population means are equal or not. However, once a factor is considered to be significant after an ANOVA procedure, there is no information regarding which population mean differs from the others. An ANOVA can inform that applying epoxy significantly affects the amount of RSD, but it does not distinguish the levels of epoxy applied. In order to differentiate the effects of low, medium or high amount of epoxy on RSD, a mean separation procedure would be necessary.

The Fisher Least Significant Difference (LSD) procedure uses t-distributions to indicate the minimum difference between two sample means that would classify the respective population means as

distinct values at a given confidence level. Classically, the results are presented in tables where letters are assigned to the means in consideration, and those values grouped with the same letter are not significantly different from each other. This is illustrated in Table 3.3 with hypothetical numbers regarding the application of epoxy in concrete crossties rail seats to reduce RSD. For the purposes of this thesis, the LSD is defined as a function of the mean square error, as described in Equation 3.13.

$$LSD = t_{\alpha/2} \sqrt{MSE\left(\frac{1}{n_i} + \frac{1}{n_j}\right)}$$
(3.13)

where,

LSD: Least significant difference.

MSE: Mean square error.

 $t_{\alpha/2}$: Critical t-value from t distribution (with error degrees of freedom).

 α : Significance level.

 n_i : Sample size from population *i*.

 n_i : Sample size from population *j*.

LSD	LSD 0.015				
	t Grouping	Mean Depth of RSD			
	t Grouping	(inches)			
No epoxy	А	0.500			
Low	А	0.490			
Medium	В	0.470			
High	С	0.450			

Table 3.3 Separation of hy	poth	etical RSD me	ans by tl	ne Fish	ner
least significant difference	(LSD)	procedure at	alpha eq	ual to	0.05

Based on the example of Table 3.2, the conclusion would be that there is no significant difference in RSD levels by applying a low amount of epoxy or not applying it at all (both have the same "t Grouping" letter). This is the case because the difference between low and no epoxy is less than the least significant difference value at alpha equal to 0.05. As it can be inferred from Equation 3.13, the LSD value would be smaller as more replicates are obtained or the confidence level decreases.

3.5 Summary

This chapter presented some statistical tools that are used within this thesis, including fundamental concepts, analysis of variance with one and two factors, and the Fisher's least significant difference. In addition to providing necessary information for readers to better understand the findings presented in this thesis, this chapter can be a stimulus for other railroad track infrastructure researchers to incorporate more statistics in their work.

CHAPTER 4: QUANTIFICATION OF BENDING MOMENTS OF NEW AND CRACKED CONCRETE CROSSTIES UNDER DIFFERENT SUPPORT CONDITIONS

4.1 Introduction

As previously discussed in Chapter 2, the ability of concrete crossties to contribute to safe and reliable railroad operations is closely related to the track support conditions. However, this understanding in the railway industry is generally qualitative, and the survey results presented in Chapter 2 revealed that there is a need to quantitatively describe the correlation between support conditions and concrete crosstie performance. Focusing on the flexural demand, which is one of the most important design considerations, this chapter attempts to quantify the influence of support conditions on bending moments of concrete crossties.

Successful mapping of concrete crosstie flexural demands to different track support conditions can lead to an improved understanding of crosstie flexural performance and more representative design requirements. Many design recommendations throughout the international railway community have adopted standardized assumptions for flexural analysis methodologies (Wolf et al., 2015), but the question of how accurately these assumptions represent revenue service track conditions remains unanswered.

Additionally, other researchers have noted that crosstie support conditions have a substantial effect on a crosstie's flexural failure mode (Yu et al., 2015) and that gaps between the ballast and the concrete crosstie at the rail seat area can result in tensile cracking along the top of the crosstie center (Chen et al., 2014). Therefore, beyond simply improving upon the design assumptions, this research can shed light on the failure modes of concrete crossties associated with their flexural behavior and inspire better maintenance practices. Ultimately, maintaining good track support can lead to a safer railroad operation, as there have been derailments where poor crosstie support conditions were determined to be a critical factor contributing to the accident (Marquis et al., 2014).

However, characterizing the actual support conditions in revenue track is nontrivial. The ballast layer is typically non-uniform and previous studies revealed significant variability in pressure distribution under concrete crossties, even between adjacent crossties with the same type of track superstructure construction (McHenry, 2013). As such, instead of a field analysis, laboratory experiments were performed to better control the variables under investigation in this research. Moreover, because concrete crossties in revenue service have been known to exhibit distress, center-cracked crossties were also tested in addition to new, un-cracked crossties. Center cracks are among the most common distresses concrete crossties exhibit, as explained in Chapter 2 of this thesis, and thus provide a representative defective condition to investigate. Using statistical methods, a detailed discussion on the resulting experimental bending moments is presented in this chapter.

4.2 Experimentation Plan

Laboratory experiments were performed by the Rail Transportation and Engineering Center (RailTEC) at the Research and Innovation Laboratory (RAIL) at the Harry Schnabel, Jr. Geotechnical Laboratory in Champaign, Illinois, to quantify the influence of support conditions on concrete crosstie bending moments. Individual concrete crossties were placed in a loading frame where both rail seats could be simultaneously loaded in the vertical direction (Figure 4.1).

The crossties were supported by rubber pads simulating various revenue service support conditions: uniform ballast layer, center binding, newly tamped track, and track with high impact wheel loads (Figure 4.2). All of the pads were 1 inch (25.4 mm) thick, 12 inches (304.8 mm) wide, and 12 inches (304.8 mm) long, with a durometer hardness of 50 shore A. The author was comfortable with the use of rubber pads as the absolute vertical displacements of the crosstie ends measured at the laboratory were in the range of 0.05 to 0.1 inches (12.7 to 25.4 mm) under a 20-kip (89 kN) rail seat load, numbers comparable to recorded displacements measured in the field (Manda et al., 2014).



Figure 4.1 (a) 3-D rendering and (b) photograph of steel loading frame with instrumented concrete crosstie



Figure 4.2 Profile view of concrete crossties showing experimental support conditions and strain gauge locations

To quantify the resulting concrete crosstie flexural demand as a result of loading and support

condition, surface strain gauges were used. Strains were measured at six locations on each crosstie tested:

two at the center of the rail seats, two at the crosstie center, and two at an intermediate point equally

distant from the rail seats and the crosstie center (Figure 4.2). One strain gauge is not shown in Figure 4.2, which was placed at the crosstie center on the opposite side relative to its longitudinal axis. Originally, the center strain gauges were placed on the top chamfer of the crossties, as shown in Figure 4.2, but their strain reading was affected when cracks developed at this location. Therefore, the center gauges had to be moved below the neutral axis of the crossties (1 inch (25.4 mm) away from the bottom in this case), in a location where the cracks did not reach. The strain gauges were manufactured by Tokyo Sokki Kenkyujo Co, Ltd. (TML) and are specifically designed for use on concrete structural elements. The gauge length and width were 1.18 inches (30.0 mm) and 0.10 inches (2.3 mm), respectively, and the gauge resistance was 120 Ohms. A primer coat and secondary coat of epoxy were used to provide a smooth surface and gauge bond, respectively. To increase the sample size and further understand the variability associated with different support conditions, the two halves of the crosstie were instrumented in a symmetric fashion (Figure 4.2). This was possible since the support and loading conditions used in this experiment were symmetric.

Both rail seats of a single crosstie were simultaneously loaded with equal vertical forces up to 20 kips (89 kN). A wheel load of 40 kips (177.9 kN) provides an approximate representation of the 95th percentile nominal wheel load for loaded freight cars in the US, based on a representative sample of railcars in unrestricted interchange on a Class I railroad (Van Dyk, 2014). A single crosstie bears approximately 50 percent of the axle load applied directly above it assuming 24 inch (610 mm) crosstie spacing (American Railway Engineering and Maintenance-of-Way Association, 2014). Therefore, loading up to 20 kips (89 kN) approximates the 95th percentile nominal rail seat load imparted by a loaded freight car in the US. From the passenger service perspective, a wheel load of 40 kips (177.9 kN) represents approximately the 90th percentile peak load of loaded commuter railcars in service on the US Northeast Corridor based on a recent study on three different commuter rail systems (Lin et al., 2016). Therefore, holding the assumption that a single crosstie bears approximately 50 percent of the axle load applied over it, loading up to 20 kips (89 kN) roughly represents the 90th percentile of peak rail seat load

that are induced by a loaded commuter rail car. For high-speed rail, however, there is limited data available on wheel loads in North America. In such case, a 13.2 kip (58.7 kN) nominal wheel load can be assumed for loaded Japanese Shinkansen rolling stock (Yanase, 2010). Considering that a speed factor of three is recommended for high-speed track design (Wang, 2015), then a design wheel load of 39.6 kips (176.1 kN) can be assumed, leading again to approximately 20 kips (89 kN) of rail seat load. Thus, the loading conditions used to collect data for this thesis are representative of nominal freight loads, dynamic commuter loads, and design high-speed rail loads.

Figure 4.2 illustrates the support conditions and strain gauge locations used for laboratory experimentation. The "full support" condition is the baseline scenario where a uniform and homogenous layer of ballast is represented by pads placed under the entire length and width of the crosstie. Two variations of "center binding" were simulated in the experiments, one being more severe than the other, which was varied by the length of pads under the crosstie center. The arrangement for "lack of rail seat support" takes into consideration the fact that, that under field conditions, the ballast below the rail seat might degrade faster due to impact loads resulting from track or wheel irregularities. Finally, the "lack of center support" configuration assumes the ballast does not provide significant support at the crosstie center area, which could represent newly tamped track. This condition is simulated by including the pads only at the area reached by the tines of the tamper.

All experiments were conducted five times with healthy concrete crossties and five times with center-cracked crossties, all of the same design. The crosstie cracks were all generated in the laboratory by simultaneously loading both rail seats of a single crosstie with equal vertical forces up to 20 kips (89 kN) while the crosstie was supported with a severe center binding condition (Figure 4.3). Typically, after cracking, each crosstie presented seven horizontal cracks that were symmetric about the crosstie midspan. All cracked crossties had cracks going deeper than the first level of presstress and the deepest cracks typically reached 3 inches (76.2 mm) of depth below the top center surface and 2 inches (50.8 mm) below the top level of presstress (Figure 4.3). It should be mentioned that when the load was removed, the

cracks closed up. However, since the cracks were deeper than the first level of presstress, the crossties were considered to be failed according to the definition set forth within the AREMA center negative bending moment test (American Railway Engineering and Maintenance-of-Way Association, 2014).



Figure 4.3 (a) Support condition used for crosstie cracking; (b) Plan view of cracks; (c) Profile view of cracks with highlighted location of first level of prestressing steel

Since each crosstie was instrumented with symmetrically-located strain gauges, ten data points were collected for each support condition for each gauge location, with healthy and cracked crossties. For statistical purposes, one replicate will be associated with half of a crosstie in this thesis, as illustrated in Figure 4.4. Therefore, ten replicates were performed for each strain gauge location, support condition, and crosstie health condition.



Figure 4.4 Strain gauge instrumentation; each half of the crosstie provides one replicate for each gauge location

Rather than present the data in strain, the results are reported as bending moments, which are more widely used and more easily interpreted than stresses or strains. The moment values were calculated based on calibration factors, which were found by applying a known moment to a crosstie with controlled load and support (Figure 4.5) and measuring the corresponding strains. The crosstie was instrumented with surface strain gauges in the same configurations as in all other experiments (Figure 4.2). These calibration procedures were executed three times.



Figure 4.5 Layout for calibrating: (a) rail seat, and (b) intermediate and center strain gauges (adapted from AREMA (American Railway Engineering and Maintenance-of-Way Association, 2014)); (c) image of loading frame used for calibration

4.3 **Results of Experimentation**

4.3.1 New Crossties

To guide the process of data analysis regarding new crossties, a statistical model was developed using the

concept of completely randomized design (CRD), as shown in Equation 1 (Ott and Longnecker, 2008).

The same model was used for the three locations (rail seat, center, and intermediate) and the load was

fixed at 20 kips (89 kN). For easier reading, Equation 4.1 uses Latin letters that are associated with their meaning (as opposed to the exclusive use of Greek letters that is typical of classical statistics):

$$m_{ij} = \mu + s_i + \varepsilon_{ij} \tag{4.1}$$

where,

- m_{ii} : jth observation of moment with the ith support condition.
- μ : Grand population mean for moment.
- s_i : Fixed effect of the ith support condition on moment.
- ε_{ii} : Random error (residual) of the jth observation with the ith support condition.

To further analyze the new crosstie experimental results with this model, the errors must meet the assumptions of being both normally and independently distributed with equal variance (Ott and Longnecker, 2008). Lack of independence is usually associated with the existence of correlation in time or space within a cluster of the model (i.e. in this case, a cluster is the set of measurements for each support condition for a given strain gauge location). As no correlation was expected to be found, the independence assumption was not formally verified. However, the other assumptions were tested and Figure 4.6 presents the probability (p-value) resulting from the Shapiro-Wilk test for normality (Shapiro and Wilk, 1965) and the Brown and Forsythe's test for homogeneity of variance (Brown and Forsythe, 1974), which are widely accepted ways of verifying such assumptions in statistics. In all three cases, the homogeneity of variance and normality assumptions were met at a significance level of at least 0.053, which was considered sufficient to meet the assumptions. For better visualization, Figure 4.6 shows the distribution of the residuals for the three cases overlapped by the closest normal distribution curves.



Figure 4.6 Probability (p-values) for statistical assumptions (Shapiro-Wilk test for normality (Shapiro and Wilk, 1965) and Brown and Forsythe's test for homogeneity of variance (Brown and Forsythe, 1974)) used) and experimental residual distributions of bending moments of new crossties compared to normal distribution curves

As previously explained, ten replicates were obtained for each support case. With this number and the highest measured mean square error (MSE), the confidence interval for the population mean was estimated using Equation 4.2, which is derived from the Central Limit Theorem (Ott and Longnecker, 2008). The deviation of the sample means relative to the respective population means is no greater than 25 kip-in (2.8 kNm) for a confidence interval of 91%.

$$n \approx \frac{(z_{\alpha/2})^2 \hat{\sigma}^2}{D^2} \tag{4.2}$$

where, *n*: Number of observations (replicates).

 $z_{\alpha/2}$: z-value from standard normal distribution.

 α : Significance level.

 $\hat{\sigma}^2$: Sample variance (MSE was used in this analysis).

D: Detectable deviation of sample mean relative to the population mean.

The experimental results of new crossties are represented in the box plots below, categorized by the support condition for the crosstie rail seat, center, and the intermediate location (Figures 4.7 to 4.9). These data show that the rail seat is nearly exclusively subjected to nonnegative bending moments regardless of the support condition (Figure 4.7). In addition, the measured bending moments are mostly below 150 kip-in (16.9 kNm). Considering that a typical design limit for rail seat positive bending moment is 300 kip-in (33.9 kNm) (American Railway Engineering and Maintenance-of-Way Association, 2014), we conclude mean moments for the most demanding experimental case, "lack of rail seat support", are 47.5% lower than this design limit. It is important to highlight that field conditions of lack of rail seat support will likely be less severe than the one used for the experiments presented in this thesis since, under field conditions, it is difficult to fully loose contact under the entire rail seat at the ballast-crosstie interface. Therefore, we can infer that rail seat cracking is not expected to occur in properly designed and manufactured concrete crossties in revenue service track under nominal wheel loads, symmetric loading, and proper support conditions. However, dynamic loads can be higher than nominal loads and the results from this experimentation are not sufficient to state that the mentioned design limit is over conservative.



Figure 4.7 Box plots for moments at the crosstie rail seat for 20-kip rail seat load

The crosstie center is primarily subjected to negative bending moments (Figure 4.8). Moreover, for high center binding, many resulting moments were larger in magnitude than 201 kip-in (22.7 kNm). This value represents a typical design limit for center negative bending moment as determined via the method described in the AREMA Manual for Railway Engineering assuming 24-inch crosstie spacing with speed and tonnage factors equal to one (American Railway Engineering and Maintenance-of-Way Association, 2014). Such high moments at the crosstie center are in agreement with the fact that center cracks are more frequent than rail seat cracks in North America, as explained in Chapter 2 of this thesis. As documented by Wolf et al. (2015), current design recommendations for center negative bending moments may need to be increased. Not surprisingly, the intermediate location between the crosstie rail seat and center is clearly a transition point for bending moments, as there are both positive and negative results at this location (Figure 4.9).



Figure 4.8 Box plots for moments at the crosstie center for 20-kip rail seat load



Figure 4.9 Box plots for moments at the crosstie intermediate location between rail seat and center for 20-kip rail seat load

In addition to the box plots, a mean separation process was implemented using the Fisher Least Significant Difference (LSD) procedure (Ott and Longnecker, 2008). Using t-distributions, this procedure indicates the minimum difference between two means that would classify them as distinct values at a given confidence level. In this work, the mean moments obtained at a rail seat load of 20 kips (89 kN) were grouped at a significance level (alpha) of 0.10 (i.e. confidence level of 0.90). The results are shown in Table 4.1, where means with the same "t Grouping" letter are not significantly different because the difference between them is less than the LSD value. At all three strain gauge locations, there was no significant difference between the moments obtained from "full support" and "lack of center support" (Table 4.1). Regarding the crosstie flexural demand, this indicates that tamping is very effective in promoting proper crosstie support conditions. In addition, at the intermediate point, the results obtained from "lack of rail seat support" are not significantly different from the ones relative to "full support" either (Table 4.1). This finding is in agreement with a previous study on the topic (Wakui and Okuda, 1997). The center bending moments, however, are similar for the cases "lack of rail seat support" and "light center binding" (Table 4.1). This finding leads to the conclusion that track experiencing high impact loads might have its support changed in such a way that both crosstie rail seat and center will have the flexural demands increased, even though the former will experience a greater absolute moment than the latter. For easier visualization, Figure 4.10 illustrates the bending moment diagram of the concrete crosstie for each support condition using the mean moment values obtained experimentally from ten replicates. It is worth noting that the "full support" and "lack of center support" curves (red with filled circle and green with asterisk) are not significantly different at any point, demonstrating the conclusions obtained with the mean separation analysis (Figure 4.10).

Table 4.1 Separation of moment means by the Fisher least significant difference (LSD) procedure at alpha equal to 0.10 (means with the same letter are not significantly different)*

Location	Rail Seat		Intermediate		Center	
LSD	12.4		23.5		35.1	
Support	t Grouping	Mean	t Grouping	Mean	t Grouping	Mean
Full Support	В	130.1	AB	33.2	A	-7.3
Light Center Binding	С	57.5	С	-35.1	В	-57.8
High Center Binding	D	-2.5	D	-172.8	С	-227.5
Lack of Rail Seat Support	А	157.5	В	26.5	В	-52.1
Lack of Center Support	В	124.0	А	52.3	А	20.8

*All values are in kip-in and correspond to a rail seat load of 20 kips (89 kN). Note: 1 kip-in = 0.113 kNm.



Figure 4.10 Bending moment diagram for new concrete crosstie under different support conditions at rail seat load of 20 kips (89 kN)

Figures 4.11, 4.12, and 4.13 show the effect of different support conditions on the crosstie mean bending moments as a function of rail seat load. As one would expect based on Euler-Bernoulli beam theory, the moments seem to behave linearly with respect to changes in rail seat load. Some curves,

however, are slightly nonlinear, which might be due to a nonlinear response of the rubber pads. However, this should be of little concern and may indeed be realistic as the ballast support might change with changes in load by closing some of the gaps between aggregate particles (Prause and Kish, 1978).

Moreover, the center negative bending moment increases in magnitude as the center binding condition becomes more severe (Figure 4.11). The slope of each curve is an indication of the sensitivity of the bending moments to the rail seat loads and it is clear that the steepest lines are associated with the center location. Figure 4.12 illustrates the effect of lack of rail seat support on bending moments. It is noticeable that, as previously mentioned, both the center and rail seat moments grow in severity when the crosstie is subjected to this type of support. Finally, Figure 4.13 confirms the results from the means separation procedure (Table 4.1), showing that the lack of center support poses no significant difference on bending moments at any location along the crosstie span when compared to the full support case.



Figure 4.11 Effect of center binding on mean values for moment (new crossties)



Figure 4.12 Effect of lack of rail seat support on mean values for moment (new crossties)





4.3.2 Center-Cracked Crossties

Similar to new crossties, a statistical model was developed including the center-cracked crosstie results. However, now the completely randomized design (CRD) had two factors, as shown in Equation 4.3 (Ott and Longnecker, 2008). The same model was used for the three locations (rail seat, center, and intermediate) and the rail seat load was fixed at 20 kips (89 kN). Once more, Equation 1 uses Latin letters that are associated with their meaning:

$$m_{ijk} = \mu + s_i + c_j + sc_{ij} + \varepsilon_{ijk} \tag{4.3}$$

where,

 m_{ijk} : kth observation of moment with the ith support condition and the jth crosstie health state.

 μ : Grand population mean for moment.

 s_i : Fixed effect of the ith support condition on moment.

 c_i : Fixed effect of the jth crosstie health state on moment.

 sc_{ij} : Effect of interaction between the ith support condition and the jth crosstie health state on moment.

 ε_{ijk} : Random error (residual) of the kth observation with the ith support condition and the jth crosstie health state.

Again, to further analyze the experimental results with this model, the errors must meet the assumptions of being both normally and independently distributed with equal variance (Ott and Longnecker, 2008). As no correlation was expected to be found, the independence assumption was not formally verified. However, the other assumptions were tested in the same way as for new crossties and the results are shown in Table 4.2. In all three cases, the homogeneity of variance was met at a significance level of at least 0.034, which was considered sufficient to meet this assumption. Conversely, the normality assumption was not met for all cases at a reasonable significance level. However, this should not be a problem, since it has often been reported that violation of the normality assumption should be of little concern (Glass et al., 1972).

Table 4.2 Probability (p Values) for Statistical Assumptions (Shapiro-Wilk test for normality (Shapiro and Wilk, 1965) and Brown and Forsythe's test for homogeneity of variance (Brown and Forsythe, 1974))

	Rail Seat	Intermediate	Center
Homogeneity of Variance	0.034	0.081	0.275
Normality	0.007	0.015	0.085

Table 4.3 summarizes the confidence interval analysis applying the MSE to Equation 4.2 and

considering the maximum deviation of the sample to population means to be 25 kip-in (2.8 kNm).

Table 4.3 Confidence for at most 25 kip-in (2.8 kNm) of deviation between the
sample mean and population mean

Rail Seat	Intermediate	Center
401	1816	2780
0.99992	0.936	0.866
	Rail Seat 401 0.99992	Rail Seat Intermediate 401 1816 0.99992 0.936

The effect of center cracks and different support conditions on bending moments were stated to be either significant or not significant based on a two-way analysis of variance (ANOVA) (Fisher, 1970). For this analysis, every strain gauge location encompassed ten factor combinations (five support conditions times two crosstie health conditions), each containing ten replicates. The null hypothesis is that all bending moment values have the same population mean. Therefore, this hypothesis implies that the effect of support and crosstie health conditions on bending moments is negligible. To reject this and state that a factor is actually significant, the probability (p-value) associated with it has to be lower than a chosen significance level (α). For this study, the null hypothesis was tested under the significance level of 0.01.

Tables 4.4, 4.5 and 4.6 present the ANOVA results for the rail seat, intermediate, and crosstie center strain gauge locations, with the last column showing the p-value ("Pr > F" column) that is

compared to the significance level. In all cases, the interaction effect is not significant, which allows for a better interpretation of the main effects (Ott and Longnecker, 2008). Not surprisingly, it is noticeable that the support condition factor has a very significant effect on bending moments. However, the crosstie health condition does not have a significant effect on the bending moments. This means that the particular cracking pattern created at the laboratory does not contribute to a significant difference in bending moments in relation to the un-cracked condition. Therefore, it is concluded that light cracks that go deeper than the first level of prestressing steel do not affect the crosstie flexural performance in terms of bending moments for the particular crosstie model and cracking pattern. However, it may be argued that such results were to be expected since, according to Euler-Bernoulli beam theory, bending moments should not change given that the loading and support conditions remain the same. Nevertheless, it is necessary to consider that large enough cracks could split the crosstie into segments that are incapable of fully transmitting flexural demands to adjacent parts due to the presence of discontinuities. This may affect the bending moment results if the prestressing steel is not capable of transmitting bending moments. In all cases, the loss of bond with the prestressing steel at the crack location would change the bending moment induced by prestressing loads.

Source	Degrees of Freedom	Sum of Squares	Mean Square	F value	Pr > F
Support Condition	4	317,206	79,302	197.8	<0.0001
Crosstie Health	1	182	182	0.5	0.50
Interaction Support-Health	4	612	153	0.4	0.82
Error	90	36,091	401		
TOTAL	99	354,091			

Table 4.4 Crosstie rail seat location ANOVA results

Source	Degrees of Freedom	Sum of Squares	Mean Square	F value	Pr > F
Support Condition	4	680,141	170,035	93.6	<0.0001
Crosstie Health	1	3,190	3,190	1.8	0.19
Interaction Support-Health	4	1,673	418	0.2	0.92
Error	90	163,455	1,816		
TOTAL	99	848,459			

Table 4.5 Crosstie intermediate location ANOVA results

Table 4.6 Crosstie center location ANOVA results

Source	Degrees of Freedom	Sum of Squares	Mean Square	F value	Pr > F
Support Condition	4	863,393	215,848	77.7	<0.0001
Crosstie Health	1	782	782	0.3	0.60
Interaction Support-Health	4	7,531	1,883	0.7	0.61
Error	90	250,193	2,780		
TOTAL	99	1,121,899			

Figure 4.14 illustrates the bending moment diagram of center-cracked concrete crossties for each support condition using the mean moment values obtained experimentally from ten replicates. The bending moments associated with the support condition used to generate the cracks (i.e. severe center binding) is also shown even though its results were not used in the statistical analysis. The same results are shown in Table 4.7, with the actual numbers being displayed.

Table 4.7 Experimental bending moments for cracked concrete crossties under different support conditions at rail seat load of 20 kips (89 kN)*

_			
	(Cracked Crosstie	;
	Rail Seat	Intermediate	Center
Full Support	123.0	49.0	5.1
Light Center Binding	63.4	-9.7	-68.5
High Center Binding	-2.7	-167.3	-246.0
Severe Center Binding	0.5	-309.0	-413.1
Lack of Rail Seat Support	150.9	32.2	-24.7
Lack of Center Support	118.5	56.5	38.0

*All values are in kip-in. Note: 1 kip-in = 0.113 kNm.



Figure 4.14 Bending moment diagram for center-cracked concrete crossties under different support conditions at rail seat load of 20 kips (89 kN)

With the purpose of making additional conclusions about the performance of deteriorated concrete crossties, it is recommended that future work examine the influences of greater cracks on bending moments. In addition, different criteria should be studied in conjunction with the bending moment analysis. The performance of cracked crossties in terms of geometry deviation focusing on gage widening may be a good complementary investigation to better evaluate degraded concrete crossties.

4.3.3 Varying Crosstie Design

All the material previously presented in this chapter is relative to the same crosstie model, which will be referred to as "Model A". In order to make conclusions that are more generally applied to different concrete crosstie designs, a different crosstie model was also tested, which will be called "Model B". The Model B crosstie was subjected to the "full support" and "severe center binding" cases, with six replicates being collected in each support condition (as opposed to the ten replicates for Model A). Both models represent widely used concrete crosstie designs in the US. In addition, both models were 8.5 feet long (2,590.8 mm) with standard gage (i.e. 56.5 inches (1,435 mm)). Crossties of Model A had 20 prestressing wires, while Model B crossties had eight prestressing strands. At the center section, Model A is 7.5 inches (190.5 mm) tall and 8.37 inches (212.6 mm) wide, while Model B is 7.0 inches (177.8 mm) tall and 10.0 inches (254.0 mm) wide. Table 4.8 summarizes the results comparing both models.

Table 4.8 Experimental bending moments for different crosstie designs under different support conditions at rail seat load of 20 kips (89 kN)*

	Rail Seat		Intermediate		Center	
	Model A	Model B	Model A	Model B	Model A	Model B
Full Support	130.1	167.6	33.2	62.3	-7.3	27.3
Severe Center Binding	0.5	-13.8	-309.0	-254.9	-413.1	-308.7

*All values are in kip-in. Note: 1 kip-in = 0.113 kNm.

It is evident that the bending moments vary as a function of the crosstie design. Crosstie Model A is stiffer, taller and thinner at the center than Model B. Such design differences affect the support reactions from the rubber pads, which makes the bending moments differ for each crosstie model, even under the same test configuration. With this in mind, extrapolation of bending moment analysis to different crosstie designs should be done with care.

4.4 Conclusions

This chapter focused on presenting laboratory experimental results that are useful in associating the flexural demand of concrete crossties with a variety of common support conditions. Moreover, the experimental loading conditions were representative of different scenarios, including nominal freight and dynamic passenger wheel loads. Understanding the performance of concrete crossties under typical support conditions is critical for accurately developing a representative mechanistic design processes for

concrete crossties. In addition to design and performance implications, this research can assist crosstie manufacturers, railroads, or researchers who would need to validate finite element (FE) models relating the structural performance of concrete crossties.

The experimental results indicated that the bending moments at the crosstie center were more sensitive to changes in support conditions than the rail seat bending moments, as shown below:

- A change of 213.1 kip-in (23.1 kNm) was observed at the crosstie center for "high center binding" compared to "full support";
- A change of 50.5 kip-in (5.7 kNm) was observed at the crosstie center for "light center binding" compared to "full support";
- A change of 27.4 kip-in (3.1 kNm) was observed at the crosstie rail seat for "lack of rail seat support" compared to "full support".

It was found that there is no statistically significant difference in bending moments from the "full support" and "lack of center support" cases. Considering that the latter may represent newly tamped track, this finding is a confirmation that tamping can be very beneficial for crossties subjected to center binding or lack of rail seat support by lowering the flexural demands placed on them. Data from this experimentation also revealed that some design recommendations might underestimate the center negative moments experienced by crossties under high center binding support condition, which could lead to crosstie cracking and, ultimately, failure. For "high center binding" and a rail seat load of 20 kips (89 kN), the mean bending moment at the crosstie center was 9.1% greater than the typical AREMA design limit. On the contrary, for the support and loading conditions tested, rail seat positive moments were lower than typical values obtained through the recommended design practices in North America. Even for the "lack of rail seat support" case, which could likely be representative of stiff track with high impact loads, the rail seat bending moment was 47.5% lower than a typical AREMA design limit. However, this evidence is not sufficient to conclude that the AREMA design limit for rail seat positive bending moment is over conservative. In fact, dynamic rail seat loads may be higher than those used in

this study and the assumption that the rail seat load is half of the wheel load is not always true (adjacent ties could have ineffective support or crosstie spacing could be greater than 24 inches). In addition, as would be expected, lack of rail seat support also induces center negative bending moments.

Additionally, evaluating the performance of cracked concrete crossties is relevant to providing safe railway operations, but it is also complex and non-trivial. By using statistical tools and bending moment results from laboratory tests, this chapter presented an evaluation of the effect of cracking on crosstie flexural performance. An analysis of variance with two factors (two-way ANOVA), namely support conditions and crosstie health conditions, indicated that:

- The particular center cracking pattern that was tested does not significantly affect crosstie bending moments even though these crossties experienced cracks that were deeper than the first level of prestressing steel;
- Crosstie bending moments are significantly affected by changes in support conditions.

Finally, it was shown that the bending moment values are dependent on crosstie design.

Therefore, results from a particular concrete crosstie model are not necessarily directly applicable to other designs.

CHAPTER 5: DETERMINATION OF THE INFLUENCES OF DETERIORATED TRACK CONDITIONS ON GAGE WIDENING IN CONCRETE CROSSTIE TRACK

5.1 Introduction

Improper track geometry can lead to accidents with potentially severe consequences, thus avoiding such conditions is a priority in ensuring safe railway operation. As discussed in Chapter 2, although concrete crossties generally hold gage better than timber crossties, gage widening derailments have still been attributed to deteriorated concrete crossties and fastening systems in the North American railway network. Further, as was discussed in Chapter 2, center cracking is one of the most critical challenges facing concrete crossties internationally. Additionally, concrete crosstie deterioration can be accelerated as tonnage accumulates and its support conditions deteriorate (i.e. ballast breaks down beneath the rail seats and no longer properly supports the crosstie). Therefore, because crosstie deterioration may make the track more prone to geometry deviations, with one such deviation being gage widening, it is important to quantify the influence of support conditions on gage widening.

To quantify the influence of support conditions on crosstie deflection and gage widening, the Rail Transportation and Engineering Center (RailTEC) performed laboratory experiments at the Research and Innovation Laboratory (RAIL) at the Harry Schnabel, Jr. Geotechnical Laboratory in Champaign, Illinois. Using a static structural loading frame, new and cracked concrete crossties were subjected to different support conditions through the use of rubber pads; the same as those used in Chapter 4. Using statistical tools, this chapter presents a discussion on the correlation between ballast support conditions and the structural health of concrete crossties, and their effect on track gage.

5.1.1 Background on Gage Widening in Railroad Track

As mentioned in Chapter 2 of this thesis, more than 25% of U.S. railway accidents on Class I timber and concrete crosstie mainlines are caused by defective infrastructure conditions, which frequently occur due

to gage widening. As illustrated in Figure 5.1, railroad track gage is the distance "measured between the heads of the rails at right angles to the rails in a plane five-eighths of an inch below the top of the rail head" (Federal Railroad Administration, 2015b). In the U.S., as in many other countries, standard track gage is 56.5 inches (1435.1 mm). The FRA regulates the allowable gage variability by track class and Table 5.1 summarizes the maximum allowable gage increase by the FRA.



Figure 5.1 Illustration of measurement of standard track gage

FRA Class of Track	Maximum allowable speed freight/ passenger trains mph (km/h)		Maximum allowable track gage increase inches (mm)	Maximum allowable change of track gage within 31 feet <i>inches (mm)</i>
Excepted	10/ -	(16/ -)	1.75 (44.45)	-
1	10/ 15	(16/ 24)	1.50 (38.10)	-
2	25/ 30	(40/ 48)	1.25 (31.75)	-
3	40/ 60	(64/ 97)	1.25 (31.75)	-
4	60/ 80	(97/ 129)	1.00 (25.40)	-
5	80/ 90	(129/ 145)	1.00 (25.40)	-
6	110/ 110	(177/ 177)	0.75 (19.05)	0.75 (19.05)
7	125/ 125	(201/ 201)	0.75 (19.05)	0.50 (12.70)
8	160/ 160	(257/ 257)	0.75 (19.05)	0.50 (12.70)
9	220/ 220	(354/ 354)	0.75 (19.05)	0.50 (12.70)

Table 5.1 FRA limits	; for increase in track dade	(Federal Railroad	Administration. 2	015b)
				/

Gage widening is typically caused by rail wear, rail roll, worn fastening systems, rail cant deficiency, or broken or bent crossties, and it contributes to wheel-drop derailments, especially in the presence of worn wheels (Wu and Wilson, 2006). With the objective of providing reference values for potential increases in track gage resulting from various track infrastructure conditions, Table 5.2 is presented, which shows that rail seat deterioration (RSD) can cause the greatest gage widening effect. However, Table 5.2 does not provide an exhaustive list of track infrastructure conditions that can lead to gage widening, given it does not account for all the conditions previously mentioned by Wu and Wilson (2006), such as rail or crosstie deflection. Even though gage widening due to one of these track infrastructure conditions would not likely cause a derailment in and of itself, the combined effect of the various conditions could lead to an accident, and therefore, it is important to account for all variables and make this table more complete.

Track Infrastructure Condition	Estimated Maximum Track Gage Increase inches (mm)	Citation
Concrete Crosstie Manufacturing Tolerance	0.0625 (1.588)	(American Railway Engineering and Maintenance-of-Way Association, 2014)
Crosstie RSD Tolerance*	1.130 (28.702)	(Choros et al., 2007)
Rail Manufacturing Tolerance	0.125 (3.175)	(American Railway Engineering and Maintenance-of-Way Association, 2014)
Rail Wear Tolerance	0.6 (15.24)	(Jeong et al., 1998)
Maximum Tolerable Rail Lateral Movement Allowed by Fastenying Systems	0.5 (12.7)	(Federal Railroad Administration, 2015)

Table 5.2 Estimate of track gage increase due to various track infrastructure conditions

*The FRA track safety standards allows for 0.5 inch of RSD (Federal Railroad Administration, 2015), which, based on (Choros 2007), could lead up to 1.13 inches of gauge widening for the worst case scenario with rail profile 136 RE.

In order for a wheel-drop derailment to occur, the track gage must be greater than some gage equivalent dimension of the wheelset. Therefore, some basic wheelset dimensions need to be understood to determine what this gage equivalent dimension is. Figure 5.2 shows the standard wheel dimensions as defined by the Association of American Railroads (Association of American Railroads, 2011). The two most relevant measurements for this analysis are the flange thickness and the rim width, which are respectively called "B" and "L" in this figure. The distance between two wheels on the same axle, measured at the back of the wheel flanges, is commonly referred as "back-to-back" dimension and will be abbreviated as "BB" in this study.





Therefore, if the track gage is greater than the combined thicknesses of both wheel rims and the back-to-back distance (L+BB+L dimension), the wheelset will never be able to rest on both rails at the same time. However, a wheel-drop derailment could happen in less severe conditions where the track gage is greater than the L+BB+B dimension, as can easily be visualized in Figure 5.3.



Figure 5.3 Critical dimensions to access the risk of gage widening derailment (adapted from (Sundaram and Sussmann, 2006))

Considering mounting and manufacturing tolerances, Table 5.3 provides reference values for dimensions L+BB+B and L+BB+L for freight and passenger cars with new, standard wide flange wheels. However, worn wheels can have flanges as small as seven eighths of an inch (22.23 mm) thick (Federal Railroad Administration, 2016a; Federal Railroad Administration, 2016b). In addition, the minimum tolerable back-to-back distance is $52^{-15}/_{16}$ inches (1344.6 mm) under field conditions (Association of American Railroads, 2008). These circumstances could result in a value L+BB+B of 59.53 inches (1512.1 mm), which is 3.03 inches (77.0 mm) greater than the standard track gage. However, this analysis does not account for the radius of the edge of the wheel (R_e in Figure 5.3), which can be as big as 0.75 inches (19.1 mm) (Association of American Railroads, 2011). By subtracting 0.75 from 3.03, the critical track gage increase would be 2.28 inches (57.9 mm), number that is slightly less than the 2.5 inches (63.5 mm) proposed by (Sundaram and Sussmann, 2006). A gage widening value of 2.28 inches (57.9 mm) is larger than what is allowed by the FRA for any track class. However, it is valuable to identify this critical track gage increase that can lead to a wheel drop derailment because accidents still happen.

Table 5.3 Wheelset mounting and manufacturing tolerances (Association of American Railroads, 2011)

Dimension* -	Nominal value inches (mm)	Minimal value inches (mm)	Maximum value inches (mm)	Difference between minimal value and standard gage <i>inches (mm)</i>	
L+BB+B	60.141 (1,527.6)	59.875 (1,520.8)	60.094 (1,526.4)	3.375 (85.7)	
L+BB+L	64.484 (1,637.9)	64.188 (1,630.4)	64.281 (1,632.7)	7.688 (195.3)	

*Dimensions are for freight and passenger cars with new standard wide flange wheels

5.2 Experimentation Plan

Laboratory experiments were performed at RAIL to quantify the influence of support conditions and light crosstie center cracking on gage widening due to concrete crosstie bending. Individual concrete crossties were placed in a steel loading frame where both rail seats could be simultaneously loaded in the vertical direction (Figure 5.4).



(a)

(b)

Figure 5.4 Rendering (a) and photograph (b) of loading frame with instrumented concrete crosstie

To quantify the vertical displacement along the crosstie span, linear potentiometers (voltage differential transducers) were used. Each crosstie was monitored with 15 potentiometers: one at the crosstie center and seven symmetrically located on each side (Figure 5.5). Therefore, there were effectively eight different locations in which vertical displacements were monitored on each half of the crosstie. Similarly, the support and loading conditions used in this experiment were always symmetric. Having both sides of the crosstie instrumented increased the sample size to further account for the variability associated with different support and crosstie conditions.



Figure 5.5 Potentiometer instrumentation; each half of the crosstie provides one replicate for each potentiometer location (except at the center)

Both rail seats of a single crosstie were simultaneously loaded with equal vertical forces up to 20 kips (89 kN). The loading conditions used are representative of nominal freight loads, dynamic commuter loads, and design high-speed rail loads, which was discussed in more detail in Chapter 4.

Figure 5.6 illustrates the support conditions used for laboratory experimentation, which were the same as those described in Chapter 4, with the same supporting rubber pad material. The "full support" condition is the baseline scenario where a uniform, homogenous layer of ballast is represented by pads under the entire crosstie. Three variations of "center binding" were simulated in the experiments, with the most severe case having the shortest length of support pads under the crosstie center. The arrangement for "lack of rail seat support" takes into consideration the fact that, under field conditions, the ballast

below the rail seat typically degrades faster than other areas under the crosstie due to impact loads resulting from track or wheel irregularities. Finally, the "lack of center support" configuration assumes that the ballast does not provide significant support at the crosstie center area, which could represent newly tamped track. This condition is simulated by including pads only at the area reached by the tines of a tamping machine.



Figure 5.6 Experimental support conditions for concrete crossties (each pad is 12 inches (304.8 mm) long)

All experiments were conducted five times with un-cracked concrete crossties and five times with center-cracked crossties, and all crossties were of the same design. The cracks were generated in the laboratory by simultaneously loading both rail seats of a single crosstie with equal vertical forces up to 20 kips (89 kN) while the crosstie was supported with a severe center binding condition (Figure 5.6). Typically, after cracking, each crosstie presented seven horizontal cracks that were approximately symmetric about its midspan. Similar to the description provided in Chapter 4, all cracked crossties had cracks extending beyond the first level of prestressing steel and the deepest cracks reached approximately 3 inches (76.2 mm) below the top surface, meaning approximately 2 inches (50.8) beyond the first level of prestressing steel. Since the cracks were deeper than the first level of prestressing steel, the crossties would be considered failed according to the AREMA center negative bending moment quality test
(American Railway Engineering and Maintenance-of-Way Association, 2014). It should be mentioned that the cracks closed up after unloading because the concrete was still prestressed. For statistical purposes, one replicate will be associated with half of a crosstie in this thesis, as illustrated in Figure 5.5. Therefore, ten replicates were performed for each potentiometer location (excluding the center potentiometer location), support condition, and crosstie health condition.

5.3 **Results of Experimentation**

In order to correlate the resulting loaded crosstie shape to a corresponding gage widening value, Equation 5.1 was derived based on basic geometry concepts, and its variables are illustrated in Figure 5.7 and Figure 5.8.



Figure 5.7 Change in track gage (Δg) due to pure crosstie bending

$$\frac{1}{2}\Delta g = \sqrt{2\left[l^2 + \frac{r^2}{4}(1+\sin^2\varphi)\right](1-\cos\theta)} \times \sin\left\{\arctan\left[\frac{l}{\frac{r^2}{4}(1+\sin^2\varphi)}\right]tan^{-1}\left(\frac{l}{r/2}\right) + \varphi - \frac{\theta}{2}\right\}$$
(5.1)
+ $\frac{w}{2}[\cos(\varphi-\theta) - \cos\varphi]$

where,

- Δg : Change of gage due to crosstie bending.
- *l*: Rail height at gage measurement location.
- r: Distance between the two potentiometers located on either side of the rail seat.
- φ : Rail cant angle.
- *w*: The width of rail head at gage measurement location.
- θ : Induced rail rotation angle:

$$\theta = \arctan\left(\frac{\Delta d}{r - \Delta d \tan \varphi + r \tan^2 \varphi}\right) \tag{5.2}$$

 Δd : Difference between the displacements of the two potentiometers located on either side of the rail seat.



Figure 5.8 Variables of Equation 5.1 used to calculate gage widening

All gage widening numbers presented in this study are based on the 136 RE rail and it is assumed that track gage is measured five eights of an inch (15.875 mm) below the top of the rail (Federal Railroad Administration, 2015b). Figure 5.9 shows the resulting gage widening for both the un-cracked crossties and cracked crossties for the different support conditions. These results demonstrate that the gage widening effect due to concrete crosstie bending can be as large as 0.103 inches (2.62 mm) for the extreme center binding support condition for this particular crosstie design. This represents 4.12% of the 2.5 inch (63.5 mm) value that has been recommended as the ultimate safety limit to avoid wheel drop derailments (Sundaram and Sussmann, 2006). In addition, it represents 6.87% and 10.30% of the FRA limits for Class 1 and Class 5 track, respectively.



Figure 5.9 Gage widening due to concrete crosstie bending at a rail seat load of 20 kips (89 kN)

To guide the process of data analysis and account for experimental variability, a statistical model was developed using the concept of completely randomized design (CRD) with two factors, as shown in Equation 5.3 (Ott and Longnecker, 2008). As was the case in Chapter 4, for easier reading, Equation 5.3

uses Latin letters that are associated with their meaning (as opposed to the exclusive use of Greek letters that is typical of classical statistics):

$$\Delta g_{ijk} = \mu + s_i + c_j + sc_{ij} + \varepsilon_{ijk}$$
(5.3)

where,

 Δg_{ijk} : kth observation of gage widening with the ith support condition and the jth crosstie health state.

 μ : Grand population mean for gage widening.

 s_i : Fixed effect of the ith support condition on gage widening.

 c_i : Fixed effect of the jth crosstie health state on gage widening.

 sc_{ij} : Effect of interaction between the ith support condition and the jth crosstie health state on gage widening.

 ε_{ijk} : Random error (residual) of the kth observation with the ith support condition and the jth crosstie health state.

As previously explained in this thesis, to analyze the experimental results with this model, the errors must be both normally and independently distributed with equal variance (Ott and Longnecker, 2008). As no correlation was expected to be found, the independence assumption was not formally verified. However, the other assumptions were confirmed using the Shapiro-Wilk test for normality (Shapiro and Wilk, 1965) and the Brown and Forsythe's test for homogeneity of variance. In order to meet them, however, the gage widening data had to be transformed (Ott and Longnecker, 2008). Due to the relationship between mean and variance in this particular dataset, the best transformation was found to be the square root of the negative natural logarithm of the data. The homogeneity of variance and normality assumptions were met at significance levels of 0.2685 and 0.1200, respectively.

As previously discussed, ten replicates were obtained for each case. With the measured mean square error (MSE), the confidence interval for the population mean was estimated using Equation 5.4, which is derived from the Central Limit Theorem (Ott and Longnecker, 2008). Using the MSE, the

deviation of the sample means relative to the respective population means is no greater than 0.01 inches (0.254 mm) for a confidence interval of 96%.

$$n \approx \frac{(z_{\alpha/2})^2 \hat{\sigma}^2}{D^2} \tag{5.4}$$

where,

n: Number of observations (replicates).

 $z_{\alpha/2}$: z-value from standard normal distribution.

 α : Significance level.

 $\hat{\sigma}^2$: Sample variance (MSE was used in this analysis).

D: Detectable deviation of sample mean relative to population mean.

The effect of center cracks and different support conditions on gage widening due to crosstie bending were stated to be either significant or not significant based on a two-way analysis of variance (ANOVA) (Fisher, 1970). For this analysis, there were twelve factor combinations (six support conditions multiplied by two crosstie health conditions), each containing ten replicates. The null hypothesis is that all gage widening values come from the same population and, consequently, have the same population mean. Therefore, this hypothesis implies that the effect of support and crosstie health conditions on gage widening due to crosstie bending is negligible. To reject the null hypothesis and state that a factor is significant instead, the probability (p-value) associated with it has to be lower than a chosen significance level (α), which has been set as 0.01 for this study

Table 5.4 presents the ANOVA results for the gage widening analysis, with the last column showing the p-value ("Pr > F" column) that is compared to the significance level. The interaction effect is not significant (p-value of 0.6017), which allows for a better interpretation of the main effects (Ott and Longnecker, 2008), as explained in more detail in Chapter 3 of this thesis. Not surprisingly, the support condition factor has a significant effect on gage widening due to crosstie bending. On the contrary, the crosstie health condition does not have a significant effect on the resulting numbers, meaning that the

particular cracking pattern created at the laboratory does not contribute to a significant difference in gage widening in relation to the un-cracked condition.

Source	DE	Sum of	Mean	E Valuo	Dr > E
	DF	Squares	Square	r value	FIF
Support	5	4.4952	0.8990	66.55	<0.0001
Cracking	1	0.0178	0.0178	1.32	0.2529
Interaction support-crack	5	0.0494	0.0099	0.73	0.6017
Error	108	1.4590	0.0135		
Corrected Total	119	6.0214			

Table 5.4 ANOVA results for gage widening analysis

Figure 5.10 shows the displacement results of un-cracked concrete crossties under the rail seat load of 20 kips (89 kips) relative to the center displacement. The highest center displacement was 0.069 inches (1.75 mm) for lack of rail seat support and the lowest was 0.039 inches (0.99 mm) for high center binding. The end displacement however, was the greatest for severe center binding (0.277 inches (7.04 mm)), and lowest for lack of center support (0.090 inches (2.27 mm)).



Figure 5.10 Absolute displacement of un-cracked concrete crossties at a rail seat load of 20 kips (89 kN)

Figure 5.11 shows the crosstie shape results (i.e. displacement relative to the center) of uncracked concrete crossties at the rail seat load of 20 kips (89 kips) for the different support conditions. As there is no statistically significant difference between un-cracked and cracked crossties, the results of the latter are not presented. It is important to highlight, however, that the initially un-cracked crossties cracked when subjected to the severe center binding condition, as explained earlier in this chapter.



Figure 5.11 Average relative displacements of un-cracked concrete crossties at a rail seat load of 20 kips (89 kN)

5.3.1 Varying Crosstie Design

All the material previously presented in this chapter is relative to the same crosstie model, which will be referred to as "Model A". In order to make conclusions that are more generally applied to different concrete crosstie designs, a different crosstie model was also tested, which will be called "Model B". The Model B crosstie was subjected to the "full support" and "severe center binding" cases, with six replicates being collected in each support condition (as opposed to the ten replicates for Model A). Both models are used widely in the US on heavy-haul freight railroad lines. In addition, both models were 8.5 feet long (2,590.8 mm) with standard gage (i.e. 56.5 inches (1,435.1 mm)). Crossties of Model A had 20 prestressing wires, while Model B crossties had eight prestressing strands. At the center section, Model A is 7.5 inches (190.5 mm) tall and 8.37 inches (212.6 mm) wide, while Model B is 7.0 inches (177.8 mm) tall and 10.0 inches (254.0 mm) wide. Table 5.5 summarizes the results comparing both models.

Table 5.5 Bending moments for two concrete crosstie designs at a				
rail seat load of 20 kips (89 kN)				

	Center Displacement inches (mm)		End Displacement inches (mm)		Gage Increase	
					inches (mm)	
	Model A	Model B	Model A	Model B	Model A	Model B
Full Support	0.051 (1.295)	0.061 (1.549)	0.082 (2.083)	0.054 (1.377)	0.024 (0.610)	-0.001 (-0.025)
Severe Center Binding	0.044 (1.118)	0.035 (0.894)	0.277 (7.036)	0.314 (7.970)	0.101 (2.565)	0.119 (3.023)

It is clear that the deflections vary as a function of the crosstie design. Even though it presented almost no gage change in the full support case, Model B allows for greater deflections, leading to a gage increase 17.8% greater than Model A for the severe center binding case, which could pose a greater risk towards a wheel drop. Conversely, stiffer crossties are more prone to cracking than the ones that allow for greater deflection, which can potentially affect crosstie life. Therefore, the design differences affect the crosstie behavior, even under the same test configuration. With this in mind, extrapolation of deflection analysis to different crosstie designs should be done with care.

In addition, it is worth mentioning that the maximum gage widening value of 0.119 inches (3.02 mm) for crosstie of Model B represents 7.9% of the FRA limit for Class 1 track, 11.9% of the FRA limit for Class 5 track and 5.2% of the ultimate safety limit of 2.28 inches (57.9 mm) recommended in section 5.1.1 of this chapter. Moreover, it can be concluded that bending of concrete crossties can induce a

greater increase in track gage than crosstie manufacturing tolerances, as shown in Table 5.6, an updated

version of Table 5.2.

Table 5.6 Estimate of track gage increase due to various track infrastructure conditions including bending of concrete crossties (updated version of Table 5.2)

	Estimated Maximum Track Gage Increase		
Track Infrastructure Condition			
	inches (mm)		
Concrete Crosstie Manufacturing Tolerance	0.0625 (1.588)		
Crosstie RSD Tolerance*	1.130 (28.702)		
Rail Manufacturing Tolerance	0.125 (3.175)		
Rail Wear Tolerance	0.6 (15.24)		
Maximum Tolerable Rail Lateral Movement Allowed by Fastenying Systems	0.5 (12.7)		
Center Bound Concrete Crosstie**	0.119 (3.02)		

*The FRA track safety standards allows for 0.5 inch of RSD (Federal Railroad Administration, 2015), which, based on Choros (2007), could lead up to 1.13 inches of gauge widening for the worst case scenario with rail profile 136 RE.

**Based on the laboratory results obtained in this research for one crosstie with no rail nor fastener. Different crosstie models might perform differently.

5.4 Conclusions

Laboratory experiments were performed to quantify the influence of support conditions and crosstie

cracking on crosstie deflection and an equation was derived to estimate the gage widening due to crosstie

bending. The primary findings from this research were:

- Center cracks that close up in the absence of load have no significant effect on change in track gage due to bending of concrete crossties (p-value of 0.25);
- Support conditions have a significant effect on the flexural performance of concrete crossties (p-value less than 0.0001);
- Concrete crosstie bending due to center binding support conditions led to a maximum gage widening of 0.119 inches (3.02 mm) for crosstie of Model B; this represents 7.9% of the FRA limit for Class 1 track, 11.9% of the FRA limit for Class 5 track, and 5.2% of the ultimate safety limit of 2.28 inches (57.9 mm) recommended in section 5.1.1 of this chapter;

- The track gage increase induced by bending of concrete crossties (maximum of 0.119 inches (3.02 mm)) can be greater than the increase induced by crosstie manufacturing tolerances (0.0625 inches (1.59 mm));
- Crosstie deflections are dependent on crosstie design (0.101 inches (2.56 mm) for Model A versus 0.119 inches (5.74 mm) for crosstie of Model B for the severe center binding case for example).

CHAPTER 6: CONCLUSIONS AND FUTURE WORK

This work is a contribution to the field of railroad track infrastructure engineering focusing on the performance of healthy and deteriorated prestressed monoblock concrete crossties under various support conditions. After identifying critical infrastructure conditions for concrete crosstie track, this thesis presented a flexural analysis of concrete crossties based on the bending moments and deflections obtained through innovative laboratory experimentation with engineered support conditions.

This thesis presented four main chapters. With the objective of gathering information to develop an effective experimental plan, Chapter 2 focused on finding the critical and relevant railroad track conditions in terms of contributing to derailments. Chapter 3 presented simple statistical tools to be used in the subsequent chapters to interpret results of laboratory experimentation. Based on experimental results, Chapter 4 discussed the influences of different support conditions and crosstie cracks on crosstie bending moments. Finally, Chapter 5 used experimental data to investigate how different support conditions and cracked crossties can cause gage widening. To conclude this work, the next sections of this chapter include a summary of findings, recommendations, and suggested future work.

6.1 Summary of Findings

Chapter 2 of this thesis presented an extensive investigation of concrete crosstie track problems that have a high frequency and severity, by reviewing relevant literature, analyzing the FRA accident database, and reporting the results of a railway industry survey. It was found that most concrete crosstie problems are not related with their functions of supporting the rails or transmitting the loads to the ballast, but with its role of restraining track gage. In addition, it was shown that concrete crosstie performance is closely related to the bending moments considered in the design of their flexural capacity. Therefore, laboratory experimentation was conducted to quantify both concrete crosstie bending moments, and track gage variability.

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6.1.1 The effect of support conditions and concrete crosstie cracking on bending moments (Chapter 4) When subjecting individual concrete crossties to a representative nominal freight railcar rail seat load and a variety of common support conditions, the results indicated that support conditions significantly affected the induced concrete crosstie bending moments. Nevertheless, there was no statistically significant difference in bending moments from the cases representing newly tamped track and fully supported track. This is confirmation that tamping can be very beneficial for crossties subjected to center binding or lack of rail seat support by lowering the flexural demands placed on them.

Furthermore, the experimental results showed that the bending moments at the crosstie center were more sensitive to changes in support conditions than the rail seat bending moments, which is in agreement with Wolf (2015). Moreover, these data revealed that some design recommendations might underestimate the center negative bending moments experienced by crossties under high center binding support condition, which could lead to crosstie cracking and, ultimately, crosstie failure. For this support condition and a rail seat load of 20 kips (89 kN), the mean bending moment at the crosstie center was 9.1% greater than the typical AREMA design limit. On the contrary, for the experimental support and loading conditions executed, rail seat positive moments were lower than typical design values obtained through the recommended design practices in North America. Even for the case that could likely be representative of stiff track with high impact loads, the rail seat bending moment was 47.5% lower than the typical AREMA design limit.

It was also found that the particular concrete crosstie cracking pattern that resulted from testing, which consisted of approximately seven center cracks that closed up in the absence of rail seat loads, does not significantly affect the crosstie bending moments even though the cracks were deeper than the first level of prestressing steel. Finally, it was shown that bending moment values are dependent on crosstie design.

6.1.2 The influence of concrete crosstie bending on track gage (Chapter 5)

With the objective of correlating crosstie deflection with safety, an equation was derived to estimate the change in track gage due to crosstie bending. Similarly, with respect to the conclusions on bending moments, center cracks that extended beyond the first level of prestressing steel, yet closed up in the absence of load, had no significant effect on change in track gage due to bending of concrete crossties. However, as was the case with the previous conclusions for bending moments, the effect of support conditions was proven significant.

Concrete crosstie bending due to center binding support conditions led to a maximum gage widening of 0.119 inches (3.02 mm). This represents 7.9% of the FRA limit for Class 1 track, 11.9% of the FRA limit for Class 5 track, and 5.2% of the ultimate safety limit of 2.28 inches (57.9 mm) recommended in Chapter 5. It also shows that the track gage increase induced by bending of concrete crossties can be greater than the increase induced by crosstie manufacturing tolerances (0.0625 inches (1.59 mm)). Lastly, it was found that crosstie deflections are dependent on crosstie design.

6.2 **Recommendations and Future Work**

The findings of this work can impact different groups related to the railway industry, including manufacturers of concrete crossties, railroads, AREMA, the FRA, and research institutions. The recommendations for these groups are discussed in this section.

When designing and manufacturing concrete crossties, it is necessary to carefully consider the effect of the expected support conditions on the flexural demand of these components. As previously shown, bending moments resulting from center binding support conditions can be higher than what some common design practices recommend. In addition, new designs should attempt to mitigate the failure modes of concrete crossties that lead to increase in track gage, including RSD, crosstie bottom deterioration, shoulder wear, and, potentially, severe center cracking.

For railroads, perhaps the most important finding of this study is that the track support has a major role in affecting crosstie behavior. Therefore, maintaining proper support conditions should be of greater concern than repairing light center cracking of concrete crossties. In addition, evaluating proper support conditions through the analysis of bending moments is very effective, and railroad track engineers can assess the flexural demands placed on a crosstie through simple installation of surface strain gauges.

When it comes to AREMA, a review of *Chapter 30 – Ties* of their manual (American Railway Engineering and Maintenance-of-Way Association, 2014) may be considered. Particularly, the recommended practices of flexural analysis for design of concrete crossties can be substantially updated to add more scientific theory to the common practices based mostly on practical experience, especially to account for the variation of support conditions. It is very likely that the rail seat cross section can be reduced without performance loss, and that the crosstie center cross section can be increased to reduce center cracking. A second point of review would be the definition of a concrete crosstie structural crack, which currently is stated as "a crack originating in the tensile face of the tie, extending to the outermost level of reinforcement or prestressing tendons and which increases in size under application of increasing load" (American Railway Engineering and Maintenance-of-Way Association, 2014). However, the work of this thesis has proven that cracks deeper than the first level of prestress generally do not affect the performance of concrete crossties, and thus are not structural in nature. Nevertheless, many concrete crosstie recommended tests included in Chapter 30 of AREMA's Recommended Practices are based on this structural crack definition, including the "Center Negative Bending Moment Test" (section 4.9.1.6), which leads such tests to have little practical meaning.

Regarding the FRA, however, additional investigation may be necessary to confirm the accuracy of the defective conditions of concrete crossties stated in the Track Safety Standards, such as the condition in which the crossties are "deteriorated to the extent that prestressing material is visible" (Federal Railroad Administration, 2015b). Even though there is still need to prove that cracks greater than the ones presented in this study can prevent crossties from performing their basic functions, it is hypothesized that even cracks that expose prestressing material may not necessarily make a crosstie defective. For instance, supposing the prestressing material is exposed at the ends of a crosstie, it is likely that the performance of that crosstie will not be significantly affected. Therefore, it is possible that portions of the regulation regarding concrete crossties in the Track Safety Standards are overly stringent.

The definition of concrete crosstie failure is far from reaching a consensus among the various groups in the railway industry, leaving research institutions with the chance of making significant scientific progress in this field. The creation of quantifiable concrete crosstie damage indices, such as proposed by Kaewunruen and Remennikov (2009), and the integration of risk analyses should also be considered. One of the primary advantages of a risk analysis is that it accounts for the potential consequences of the incidents under analysis, such as the creation of track geometry irregularities, instead of simply condemning cracked crossties that may have only aesthetic imperfections. Nonetheless, additional research is needed to quantify the influence of more severely deteriorated conditions of concrete crossties and its components can also be of great assistance to reduce the necessary amount of laboratory and field experimentation in this investigation.

REFERENCES

- American Railway Engineering and Maintenance-of-Way Association. 2014. Chapter 30 Ties. AREMA Manual for Railway Engineering, AREMA, Landover, Maryland.
- Association of American Railroads. 2008. *Field Manual of the AAR Interchange Rules*. Association of American Railroads, Washington, D.C.
- Association of American Railroads. 2011. Section G Wheels and Axle Manual. *AAR Manual of Standards and Recommended Practices*, Association of American Railroads, Washington, D.C.
- Brown, M.B. and A.B. Forsythe. 1974. Robust tests for the equality of variances. *Journal of the American Statistical Association* 69 (346): 364–367.
- Chen, Z., M. Shin, B. Andrawes and J.R. Edwards. 2014. Parametric study on damage and load demand of prestressed concrete crosstie and fastening systems. *Engineering Failure Analysis* 46: 49–61.
- Choros, J., M.N. Coltman and B. Marquis. 2007. Prevention of Derailments Due To Concrete Tie Rail Seat Deterioration. In: *Proceedings of JRCICE2007*, Pueblo, Colorado, USA, January 2007, pp. 173–181.
- Clouse, A.L. 2012. New Concrete Crosstie Regulations and Rail Cant Measurement. , 2012, pp. 27–36.
- Dick, C.T. 2001. Factors Affecting the Frequency and Location of Broken Railway Rails and Broken Rail Derailments. M.S. Thesis. University of Illinois at Urbana-Champaign.
- Domingo, L.M., C. Zamorano Martin, C. Palenzuela Aviles and J.I. Real Herraiz. 2014. Analysis of the influence of cracked sleepers under static loading on ballasted railway tracks. *The Scientific World Journal* 2014: 363547.
- Durbin, J. and G.S. Watson. 1950. Testing for Serial Correlation in Least Squares Regression. I. *Biometrika* 37 (3-4): 409–428.
- Durbin, J. and G.S. Watson. 1951. Testing for Serial Correlation in Least Squares Regression. II. *Biometrika* 38 (1-2): 159–178.

Ellingwood, B. and T.V. Galambos. 1982. Probability-based criteria for structural design. *Structural Safety* 1 (1): 15–26.

Federal Railroad Administration. 2011. FRA Guide for Preparing Accident/Incident Reports.

Federal Railroad Administration. 2015a. Railroad Accidents/Incidents: Reports Classification, And Investigations.

Federal Railroad Administration. 2015b. Track Safety Standards.

Federal Railroad Administration. 2016a. Railroad Freight Car Safety Standards.

Federal Railroad Administration. 2016b. Railroad Locomotive Safety Standards.

- Fisher, R.A. 1970. *Statistical Methods for Research Workers*, 14th revised and enlarged ed. Oliver and Boyd, Edinburgh.
- Glass, G.V., P.D. Peckham and J.R. Sanders. 1972. Consequences of failure to meet assumptions underlying the fixed effects analyses of variance and covariance. *Review of educational research*: 237–288.
- Grassie, S.L. and S.J. Cox. 1984. Dynamic response of railway track with flexible sleepers to high frequency vertical excitation. *Proceedings of the Institution of Mechanical Engineers, Part D: Journal of Automobile Engineering* 198 (7): 117–124.
- Greve, M., M.S. Dersch, J.R. Edwards, C.P. Barkan, H. Thompson, T. Sussmann and M. McHenry. 2015.
 Examination of the Effect of Concrete Crosstie Rail Seat Deterioration on Rail Seat Load
 Distribution. In: *Transportation Research Board 94th Annual Meeting*, 2015.
- Harris, D., R. Lutch, T. Ahlborn and P. Duong. 2011. Optimization of a prestressed concrete railroad crosstie for heavy-haul applications. *Journal of Transportation Engineering* 137 (11): 815–822.
- Kaewunruen, S. and A.M. Remennikov. 2006. Sensitivity analysis of free vibration characteristics of an in situ railway concrete sleeper to variations of rail pad parameters. *Journal of Sound and Vibration* 298 (1-2): 453–461.

- Kaewunruen, S. and A.M. Remennikov. 2009. Progressive failure of prestressed concrete sleepers under multiple high-intensity impact loads. *Engineering Structures* 31 (10): 2460–2473.
- Kaewunruen, S. and A.M. Remennikov. 2010. Dynamic crack propagations in prestressed concrete sleepers in railway track systems subjected to severe impact loads. *Journal of Structural Engineering* 136 (6): 749–754.
- Kernes, R. 2014. *The Mechanics of Abrasion on Concrete Crosstie Rail Seats*. M.S. Thesis. University of Illinois at Urbana-Champaign.
- Leong, J. 2007. Development of a limit state design methodology for railway track.
- Lin, X., R. Edwards, Marcus Dersch and C. Ruppert Jr. 2016. Load Quantification for Light Rail, Heavy Rail, and Commuter Rail Transit Infrastructure.
- Lutch, R.H., Devin K. Harris and Theresa M. Ahlborn. 2009. Prestressed Concrete Ties in North America. In: *AREMA 2009*, 2009.
- Manda, K.R., M. Dersch, R. Kernes, R.J. Edwards and D.A. Lange. 2014. Vertical Load Path under Static and Dynamic Loads In Concrete Crosstie and Fastening Systems. In: 2014 Joint Rail Conference, American Society of Mechanical Engineers, 2014.
- Marquis, B., J. LeBlanc, H. Yu and D. Jeong. 2014. *Volpe Report on CSX Train Derailment on MN Tracks*. National Transportation Safety Board, United States.
- Marquis, B.P., M. Muhlanger and D.Y. Jeong. 2011. Effect of Wheel/Rail Loads on Concrete Tie Stresses and Rail Rollover. In: *Proceedings of the ASME 2011 Rail Transportation Division Fall Technical Conference*, Minneapolis, Minnesota, USA, January 2011, pp. 143–150.
- Mayville, R.A., Liying Jiang and Matthew Sherman. 2014. *Performance Evaluation of Concrete Railroad Ties on the Northeast Corridor*. Federal Railroad Administration, Washington DC.
- McHenry, M. 2013. Pressure Measurement At The Ballast-Tie Interface Of Railroad Track Using Matrix Based Tactile Surface Sensors. M.S. Thesis. University of Kentucky.

- Murray, M. 2015. Heavy Haul Sleeper Design A Rational Cost-saving Method. In: *IHHA 2015 Conference*, Perth, Australia, June 2015.
- National Transportation Safety Board. 2006. *Derailment of Amtrak Train No.* 27. National Transportation Safety Board, Washington, D.C.
- Ott, R. and M. Longnecker. 2008. An Introduction to Statistical Methods and Data Analysis. Cengage Learning.
- Prause, R.H. and A. Kish. 1978. Statistical description of service loads for concrete crosstie track. *Transportation Research Record* (694).
- Rezaie, F., M.R. Shiri and S.M. Farnam. 2012. Experimental and numerical studies of longitudinal crack control for pre-stressed concrete sleepers. *Engineering Failure Analysis* 26: 21–30.
- Robert O. Kuehl. 1999. Design of Experiments Statistical Principles of Research Design and Analysis, 2nd ed.
- Shapiro, S.S. and M.B. Wilk. 1965. An analysis of variance test for normality (complete samples). *Biometrika*: 591–611.
- Shurpali, A.A., J.R. Edwards, R.G. Kernes, D.A. Lange and C.P. Barkan. 2013. Investigation of Material Improvements to Mitigate the Effects of the Abrasion Mechanism of Concrete Crosstie Rail Seat Deterioration. *Journal of Transportation Engineering* 140 (2): 04013009.
- Sundaram, N. and T. Sussmann. 2006. Development of Gage Widening Projection Parameter for the Deployable Gage Restraint Measurement System. Federal Railroad Administration, Washington, DC.
- Taherinezhad, J., M. Sofi, P.A. Mendis and T. Ngo. 2013. A review of behavior of prestressed concrete sleepers. Advances in Rail Track Infrastructure Research and Practice (ARTIRP)-2013 13 (1): 1– 16.
- Thun, H., S. Utsi and L. Elfgren. 2008. Load carrying capacity of cracked concrete railway sleepers. *Structural Concrete* 9 (3): 153–161.

- Van Dyk, B. 2014. *Characterization of the Loading Environment for Shared-use Railway Superstructure in North America*. M.S. Thesis. University of Illinois at Urbana-Champaign.
- Van Dyk, B.J., M.S. Dersch and J.R. Edwards. 2012. International Concrete Crossile and Fastening System Survey–Final Results. Federal Railroad Administration.
- Wakui, H. and H. Okuda. 1997. A Study on Limit State Design Method for Prestressed Concrete Sleepers. In: *Proceedings-Japan Society of Civil Engineers*, DOTOKU GAKKAI, 1997, pp. 35– 54.
- Wang, P. 2015. Design of High-Speed Railway Turnouts: Theory and Applications. Academic Press.
- Wolf, H.E. 2015. *Flexural Behavior of Prestressed Concrete Monoblock Crossties*. University of Illinois at Urbana-Champaign, Urbana, IL, USA.
- Wolf, H.E., J.R. Edwards, M.S. Dersch and C.P. Barkan. 2015. Flexural Analysis of Prestressed Concrete Monoblock Sleepers for Heavy-haul Applications: Methodologies and Sensitivity to Support Conditions. In: *International Heavy Haul Association Conference*, International Heavy Haul Association, Perth, Australia, June 2015, pp. 393–402.
- Wu, H. and N. Wilson. 2006. 8 Railway Vehicle Derailment and Prevention. Handbook of railway vehicle dynamics: 209.
- Yanase, N. 2010. High Speed Rolling Stock in Japan.
- Yu, H., D. Jeong, J. Choros and T. Sussmann. 2011. Finite Element Modeling Of Prestressed Concrete Crossties with Ballast and Subgrade Support. In: ASME 2011 International Design Engineering Technical Conferences and Computers and Information in Engineering Conference, American Society of Mechanical Engineers, 2011, pp. 1077–1086.
- Yu, H., D. Jeong, B. Marquis and M. Coltman. 2015. Railroad Concrete Tie Failure Modes and Research Needs. In: 2015 Transportation Research Board 94th Annual Meeting, TRB15-0311, 2015.
- Zeman, J.C. 2010. *Hydraulic Mechanisms of Concrete-Tie Rail Seat Deterioration*. M.S. Thesis. University of Illinois at Urbana-Champaign.

 Zeman, J.C., J.R. Edwards, D.A. Lange and C.P. Barkan. 2010. Investigation of Potential Concrete Tie Rail Seat Deterioration Mechanisms: Cavitation Erosion and Hydraulic Pressure Cracking. In: *Proceedings of the Transportation Research Board 89th Annual Meeting, Washington, DC*, 2010.
 ZETA-TECH. 2010. Assessment of Concrete Tie Life on US Freight Railroads. Railway Tie Association.

APPENDIX A: SAS CODE FOR STATISTICAL ANALYSIS

The statistical analysis of this thesis was conducted using the SAS software package. All the relevant code for both bending moments and gage widening analysis are included in this appendix.

A.1 Bending Moments

```
PROC IMPORT OUT=SG
              DATAFILE="/Strain Gauges/Results_for_SAS-SG.xlsx" DBMS=xlsx
              REPLACE;
       SHEET="Moments";
       GETNAMES=YES;
RUN;
Data FRA:
       Project='FRA';
run;
DATA SG1;
       set SG(where=(Location=1 and Load=20));
run;
DATA SG2;
       set SG(where=(Location=2 and Load=20));
run;
DATA SG3;
       set SG(where=(Location=3 and Load=20));
run;
proc print data=FRA;
       title 'COMBO SG1 Load20';
run;
proc glm data=SG1;
       class Rep Combo;
       model Moment=Combo;
       output out=Resids r=residual;
       means Combo/hovtest=bf;
       run;
proc print data=FRA;
       title 'COMBO SG2 Load20';
run;
proc glm data=SG2;
       class Rep Combo;
```

model Moment=Combo; output out=Resids r=residual; means combo/hovtest=bf; run; proc print data=FRA; title 'COMBO SG3 Load20'; run; proc glm data=SG3; class Rep Combo; model Moment=Combo; output out=Resids r=residual; means combo/hovtest=bf; run; proc print data=FRA; title 'SG1 Load20'; run; proc glm data=SG1; class Rep Support Crack; model Moment=Support Crack Support*Crack; output out=Resids r=residual; means Support Crack/ lsd alpha=0.1; run; proc means data=SG1 mean q1 median q3 stddev; var Moment; class Support; run; proc univariate data=resids normal plot; var residual; run; quit; proc print data=FRA; title 'SG2 Load20'; run; proc glm data=SG2; class Rep Support Crack; model Moment=Support Crack Support*Crack; output out=Resids r=residual; means Support Crack/ lsd alpha=0.1; run:

proc means data=SG1 mean q1 median q3 stddev;

```
var Moment;
       class Support;
run;
proc univariate data=resids normal plot;
       var residual;
run;
quit;
proc print data=FRA;
       title 'SG3 Load20';
run;
proc glm data=SG3;
       class Rep Support Crack;
       model Moment=Support Crack Support*Crack;
       output out=Resids r=residual;
       means Support Crack/ lsd alpha=0.1;
       run;
proc means data=SG1 mean q1 median q3 stddev;
       var Moment;
       class Support;
run;
proc univariate data=resids normal plot;
       var residual;
run;
quit;
A.2
       Gage Widening
PROC IMPORT OUT=GW
              DATAFILE="/Potentiometers/Results_for_SAS-Pot.xlsx"
              DBMS=xlsx REPLACE;
       SHEET="Potentiometers";
       GETNAMES=YES;
RUN;
Data FRA;
       Project='FRA';
run;
```

```
DATA GW;
set GW(where=(Load=20 and Location=3 and Crack~=2));
run;
```

```
DATA GW;
       set GW;
       TGaugeChange=sqrt(-log(GaugeChange));
       %*The natural logarithm (base e) - Means are related to standard
               deviation;
run;
proc print data=FRA;
       title 'Combo';
run;
proc glm data=GW;
       class Rep Combo;
       model TGaugeChange=Combo;
       output out=resids r=residual;
       means Combo/ hovtest=bf;
       run;
proc print data=FRA;
       title 'Gauge Widening';
run;
proc glm data=GW plots=all;
       class Rep Support Crack;
       model TGaugeChange=Support Crack Support*Crack;
       output out=resids r=residual;
       means Support Crack/ lsd alpha=0.1;
       run;
proc means data=GW mean var q1 median q3 stddev;
       var TGaugeChange;
       class Crack Support;
run;
proc univariate data=resids normal plot;
       var residual;
run;
quit;
```