CHARACTERIZATION OF THE LOADING ENVIRONMENT FOR SHARED-USE RAILWAY SUPERSTRUCTURE IN NORTH AMERICA

BY

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THESIS

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ABSTRACT

A sustained increase in heavy axle loads and cumulative freight tonnages, coupled with increased development of high speed passenger rail, is placing an increasing demand on railway infrastructure. Some of the most critical areas of the infrastructure in need of further research are track components used in high speed passenger, heavy haul, and shared infrastructure applications. In North America, many design guidelines for these systems use historical wheel loads and design factors that may not necessarily be representative of those seen on rail networks today. Without a clear understanding of the nature of these loads, it is impossible to adequately evaluate the superstructure to make design improvements. Therefore, researchers at the University of Illinois at Urbana-Champaign (UIUC) are conducting research to lay the groundwork for an improved and thorough understanding of the loading environment entering the track structure. Multiple wheel load measurement technologies have been used historically to monitor vehicle health by measuring and recording information related to vertical, lateral, and longitudinal loading. This information can be used to identify and classify trends in the loading spectrum and other characteristics of the rolling stock. These trends not only provide a clearer picture of the existing loading environment created by widely varied traffic characteristics, but can be used in future design and maintenance planning activities according to the anticipated traffic. This thesis will discuss the current trends in the loading environment across the North American rail network while investigating the effects of speed and other sources of variability. Ultimately this work should lead to useful distinctions of loads for evaluating and improving design methodologies that are based on current loading conditions.

Dedicated to the One from whom all blessings flow

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CHAPTER 1: INTRODUCTION

1.1 Purpose

The purpose of this research was to characterize the loading environment of shared-use railway infrastructure to improve the design and performance of critical superstructure components.

1.2 Motivation and Background

There are approximately 25 million concrete crossties in track on heavy haul lines in North America. They are typically installed where timber crossties cannot perform satisfactorily in certain severe service conditions, such as high curvature, heavy axle load freight traffic, high-speed passenger traffic, high annual gross tonnages, steep grades, and severe climatic conditions including areas of high moisture (Rapp et al. 2013). Many of these crossties and other elements of the track superstructure in North America have historically been designed through a process that is generally based on practical experience without a clear analytical understanding of the loading environment leading to particular failure mechanisms. The analytical process that is used in the current design recommendations uses loading information that is highly variable and not necessarily representative of today's operating conditions. Therefore, to economically optimize and improve the performance of these critical infrastructure components through enhanced design recommendations, the loading environment existing within today's rail networks must be more thoroughly characterized.

Historically, some efforts have been undertaken to better understand the loading environment throughout the entire crosstie and fastening system (Van Dyk et al. 2013c). Results from an international survey conducted in 2012 (Chapter 2) provide a summary of the current state of practice for concrete crossties and elastic fastening systems, providing direction and motivation for research related to understanding the loading demands on track components and how they related to particular failures within the system. Additional research has been performed to better understand the wheel-rail interface, especially with regards to dynamic and impact loading. This is accomplished, in part, by careful analysis of data from existing systems designed to monitor the performance of rail rolling stock.

The wheel impact load detector (WILD) was first installed on Amtrak's Northeast Corridor (NEC) in 1983 to provide information regarding potentially crack-inducing wheel loads (Moody 1987). The use of the WILD, and subsequent reduction in severe impact wheel loads, caused a reduction in concrete crosstie cracking, a sharp decline in axle bearing problems (Moody 1987), and a reduction in energy and maintenance costs (Acharya et al. 1993). A review of WILD data determined that this technology consistently and reliably recorded accurate impact load events (Wiley and Elsaleiby 2007). Amtrak utilized this detection technology to monitor the passenger and freight rolling stock traveling over its network (Trosino 2011). The data showed that the highest impact loads did not occur due to the fastest or heaviest vehicles and that the peak load was largely dependent on the wheel quality. Union Pacific Railroad (UPRR) has installed WILDs strategically throughout its network to protect their infrastructure, evaluate equipment performance, and identify load distribution issues (GeMeiner 2005). Data collected by UPRR over multiple years have shown that the number of severe impact loads increases during the winter months.

Researchers in Finland analyzed data from three different WILD designs on the same line in summer and winter conditions (Nurmikolu et al. 2013). While the three types of detectors produced reliable static data, the dynamic loads were more inconsistent, pointing toward the importance of careful dynamic calibration for an individual detector. Similar to UPRR's experience, Finnish winter conditions increased impact loads, but increased speed only produced slightly higher impact loads, on average.

Perhaps the most applicable utilization of impact load data to quantify loads at the wheel-rail interface was conducted by researchers in Australia. A new design approach has been developed that incorporates a probabilistic analysis of impact loads to improve the performance of concrete crossties (Remennikov et al. 2008). An extensive data collection and processing methodology was used to form the basis of a design load environment necessary for this improved design approach (Leong 2007). It was determined that speed, vehicle operator behavior, and maintenance practices are major contributors to the prediction of probability and return periods associated with particular impact loads (Leong 2007).

Once the wheel load is determined, that load must be traced throughout the supporting track superstructure. There are several analytical methods that estimate the distribution of loads at various interfaces within the system, many of which are similar, with some differing substantially. A mechanistic approach to understanding the loading environment that uses scientifically sophisticated relationships, complimented with data collected from realistic field conditions, will contribute to the development of improved designs, resulting in safer and more efficient rail transportation.

The previous research performed by railroad owners and institutions around the world provide a foundation for the work contained within this thesis. Much of the content within this thesis builds on those previously executed research efforts.

1.3 Objectives

An improved understanding of the loading environment entering the track structure can lead to improvements in design and, subsequently, performance of critical infrastructure components. By analyzing trends on use and performance internationally and comparing various design methodologies, enhanced design recommendations can be developed for the North American railroad superstructure. Implementing the actual North American loads within the design process using existing and new factor processes (particularly as they relate to dynamic and impact loading) will improve the effectiveness of design, leading to enhanced safety and decreased infrastructure component life cycle costs.

1.4 Thesis Organization

After a comprehensive review of an international survey conducted by the University of Illinois at Urbana-Champaign (UIUC) focusing on the design and performance of concrete crossties and elastic fastening systems (Chapter 2) and a discussion on an improved design process for these infrastructure components (Chapter 3), the focus of this thesis will discuss sources of load variation on the North American rail network (Chapter 4) and how this variation is considered in the design of the track superstructure (Chapter 5). It will then turn toward lateral and longitudinal loading characteristics and a future statistical loading environment model (Chapter 6).

This thesis is comprised of six chapters, including an introduction, conclusion, and four sections within the body wherein the following questions are to be answered:

1. What types of design and performance decisions are made internationally related to the concrete crosstie and elastic fastening system? (Chapter 2)
2. How can the current infrastructure design methodology be improved through a more mechanics-based approach? (Chapter 3)
3. What technology can be used to measure wheel-rail forces and how do they show causes of wheel load variation? (Chapter 4)
4. How have the load variation sources been included historically in the infrastructure design process and how well do these methods represent today's loading? (Chapter 5)

Chapter 2:

Presented in part at the 2012 Joint Rail Conference (JRC) in Philadelphia, Pennsylvania and submitted in part as a chapter within the FRA Tie and Fastener BAA Final Report (Edwards et al. 2014)

Improving the design and performance of concrete crossties and elastic fastening systems in North America requires a thorough understanding of the design criteria and performance trends for these components both domestically and internationally. There is substantial concrete crosstie and elastic fastening system experience internationally, but many of the operating environments and manufacturing processes differ greatly from that in North America, resulting in different failure trends and research strategies. To adequately apply improved understanding relating to the loading and failure effects to design of critical infrastructure components, a proper state of practice must be established based on information gathered from the international railway community.

Chapter 3:

Presented in part at the 2013 International Heavy Haul (IHHA) Conference in New Delhi, India and published in the conference proceedings (Van Dyk et al. 2013a)

A limited understanding of the complex loading conditions affecting the concrete crosstie and elastic fastening system components led to a design process based primarily on practical experience and previous techniques, which fails to include key variables that relate to actual field loading conditions. This process, which is typically driven by production and installation economics, has generated components that are over-designed or do not achieve their design life. The use of field and laboratory experimental data, as well as complete concrete crosstie and fastening system analytical modeling, can be used to improve the current understanding of the loading demands on each component within the system. Improved mechanistic design recommended practices for concrete crosstie and elastic fastening system design will contribute to improved safety, reliability, and rail capacity.

Chapter 4:

Presented in part at the 2013 JRC in Knoxville, Tennessee and published in the conference proceedings (Van Dyk et al. 2013b)

Many design guidelines for track components used in high speed passenger, heavy haul, and shared infrastructure applications use historical wheel loads that may not necessarily be representative of those seen on rail networks today. A more thorough understanding of the loading environment entering the track structure is necessary to adequately evaluate the superstructure and make design improvements. Information regarding loads obtained from wheel impact load detectors (WILDs) and instrumented wheel sets (IWSs) can be used to identify trends that provide a clearer picture of the existing loading environment created by widely varied traffic characteristics and to improve future design and maintenance planning of infrastructure according to the anticipated traffic.

Chapter 5:

In North America, many design guidelines for track components in shared-use railway infrastructure use historical wheel loads and many factors. To evaluate the components found in the superstructure and make design improvements, the nature of these loads and how the design process reflects them must be thoroughly understood. Design factors that have been developed internationally are assessed based on wheel loads using several existing and new evaluative metrics. New design factors are also developed to accurately represent the wheel loading environment, taking into consideration peak vertical load and the frequency at which those loads are imparted. An evaluative approach to historical and innovative design methodologies will provide improvements to design based on actual loading experienced on today's rail networks.

The content of these chapters include the portions of work completed as part of the FRA Tie and Fastener BAA most applicable to the purpose as stated in Section 1.1. Please refer to the currently unpublished version of the final report written for the FRA (Edwards et al. 2014) and downloadable resources found at <http://ict.uiuc.edu/railroad/CEE/crossties/downloads.php> for a comprehensive narrative of a substantial portion of UIUC's concrete crosstie and elastic fastening system research program and findings.

CHAPTER 2: INTERNATIONAL CONCRETE CROSSTIE AND FASTENING SYSTEM SURVEY – FINAL RESULTS[1]

2.1 International Concrete Crosstie and Fastening System Survey Objectives

The primary objective of the International Concrete Crosstie and Fastening System Survey (hereafter referred to as the "International Survey") was to poll the international railway community on the use and performance of concrete crossties and elastic fastening systems. The survey has aided UIUC's research team in developing an understanding of the most common crosstie and fastening system failures, as well as the current state-of-practice regarding the design and maintenance of these systems. Finally, it has enabled UIUC to continue establishing relationships and encourage collaboration with railways, researchers, and manufacturers around the world.

The International Survey provides insight to guide many aspects of the FRA Tie and Fastener BAA project at UIUC (including modeling and laboratory and field experimentation), ultimately leading toward improved design recommendations for concrete crossties and elastic fastening systems. In terms of modeling, results from this survey can help determine typical loading scenarios using modeling and loading methodologies from previous research. The survey results relating to modeling also provide references for literature related to previous analysis, allowing UIUC's team to incorporate past research efforts and findings into its current work. The responses from the survey also include criteria from laboratory testing performed on concrete crossties and elastic fastening systems around the world, offering the ability to compare North American test criteria and methodologies with multiple international standards. Finally, the survey results help steer the field experimentation efforts by identifying conditions where failure most commonly occurs and developing a greater understanding of probabilistic loading conditions and failure modes.

[1] Chapter 2 will be published as part of the 2010 FRA Tie and Fastener BAA Final Report by the University of Illinois at Urbana-Champaign (Edwards et al. 2014)

2.2 Audience

The International Survey was distributed to professionals in many different positions and organizations within the railroad industry, including infrastructure owners, operators, or maintainers; academic, industry, or institutional researchers; and concrete crosstie or fastening system manufacturers. This breadth of coverage provides varied perspectives on the usage and performance of concrete crossties and elastic fastening systems. Additionally, the survey's audience was geographically diverse, with responses from the international railway community in Asia, Australia, Europe, and North America.

2.3 Development

The International Survey was developed with extensive input from many of the North American experts in concrete crosstie and elastic fastening system design, production, use, maintenance, and research. First, a list of questions was developed internally at UIUC regarding the design, usage, performance, and failure of concrete crossties and elastic fastening systems. After researching various online survey tools and creating an initial test survey, the questions were distributed to the UIUC FRA Tie and Fastener BAA Industry Partners, FRA, and UIUC research team for review and subsequent revision. The industry partners, who include experts in concrete crosstie and elastic fastening system design and performance in North America, provided feedback based on North American railroading experience and what the rail industry would like to gain from such a survey. After a substantial modification and revision period, the survey was distributed to the international railway community using the online survey tool Zoomerang.

A separate set of questions was distributed to fastening system manufacturers, and was addressed during subsequent personal conversations. This facilitated more comprehensive answers regarding the fastening system landscape. A summary of these responses are included in Section 2.5.2.

2.4 Content

The content of the International Survey, which includes many aspects of the system's production, performance, and research, can be explored by seeing the comprehensive question and response lists found in Appendices A, B, and C. The appendices include the following:

Appendix A – Infrastructure Owner, Operator, or Maintainer

Appendix B – Academic, Industry, or Institutional Researcher

Appendix C – Concrete Crosstie Manufacturer

2.5 Results

2.5.1 General Survey Responses

The survey was distributed to individuals at 46 organizations who the authors believed to have extensive knowledge of the performance and design of concrete crossties and elastic fastening systems within their organization and/or their country. Of those 46 organizations invited to participate in the survey, 28 responses were received, which corresponds to a 61% response rate.

Responses were received from Asia (five responses), Australia (five), Europe (eight), and North America (ten). Nine respondents were infrastructure owners, operators, or maintainers, twelve were academic, industrial, or institutional researchers, and seven were concrete crosstie manufacturers. Given the breadth of international expertise that was captured, the number of responses was considered appropriate for achieving the objectives of this survey. Although there were no responses from Africa or South America, the authors feel that the responses are representative of the concrete crosstie and elastic fastening system community internationally.

2.5.2 General Survey Results

In the development of revised design recommendations, it is important to consider failure mechanisms and field performance of components and systems. Causes of failure provide guidance for improvement of the concrete crosstie and elastic fastening system. The most common failure causes as expressed by the responses are fastening system wear and damage, tamping damage, and concrete deterioration beneath the rail (although many of the international researchers viewed this as the least critical failure cause). It should also be noted that structural failures are viewed as critical problems by the infrastructure owners and researchers, but are not considered to be very significant relative to other failures according to the crosstie manufacturers. Figure 2.1 and Tables 2.1 and 2.2 communicate some of the key findings concerning failure.

Figure 2.1 depicts the criticality of concrete crosstie and elastic fastening system problems from most to least critical, as expressed by the international and North American respondents.

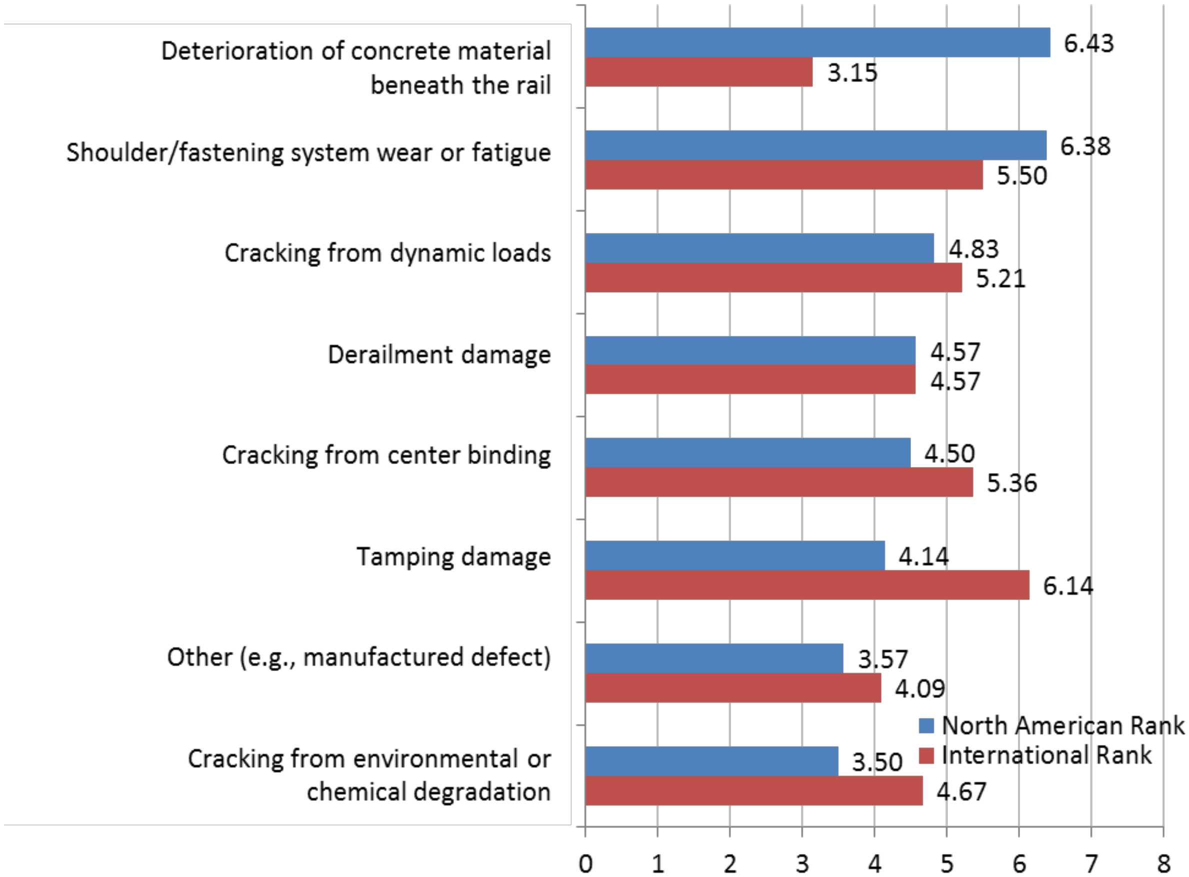

Figure 2.1 The most critical concrete crosstie and elastic fastening system problems; ranked from 1 to 8, with 8 being the most critical

Wear and fatigue in the shoulder and other components of the fastening system were determined to be critical problems, according to both international and North American respondents. The international respondents expressed tamping damage as being their most critical problem, which could indicate that, comparatively, the other potential problems are not viewed as very critical. This response could also indicate that there is damage due to the tamping process or caused by infrequent or insufficient tamping. In North America, the most critical problem was determined to be rail seat deterioration (RSD). This was in sharp contrast with the international respondents, who ranked RSD as being the least critical problem.

Table 2.1 shows the summation of the North American responses indicating failures resulting in deficiencies of concrete crossties and elastic fastening systems. For example, 71% of respondents indicated that concrete deterioration beneath the rail seat was a failure mechanism that was associated with their operating environment. Table 2.2 provides the same information according to the international respondents.

Table 2.1 The most prevalent failure causes resulting in concrete crosstie and elastic fastening system deficiencies according to North American responses

Failure Causes Resulting in Deficiencies	Percentage of Responses (%)
Concrete deterioration beneath the rail	71
Fastening system damage	43
Poor bonding of concrete to prestress	43
Poor material quality or behavior (of clamp, insulator, rail pad, or crosstie)	29
Poor environmental conditions (e.g. moisture or fines intrusion)	29
Manufacturing flaws	29
Improper component design (of clamp, insulator, rail pad, or crosstie)	29
Deficient concrete strength	14
Improper prestress force	14
Other	14

Table 2.2 The most prevalent failure causes resulting in concrete crosstie and elastic fastening system deficiencies according to international responses

Failure Causes Resulting in Deficiencies	Percentage of Responses (%)
Fastening system damage	50
Poor material quality or behavior (of clamp, insulator, rail pad, or crosstie)	44
Manufacturing flaws	44
Improper component design (of clamp, insulator, rail pad, or crosstie)	38
Concrete deterioration beneath the rail	38
Poor environmental conditions (e.g. moisture or fines intrusion)	31
Other	31
Poor bonding of concrete to prestress	25
Deficient concrete strength	19
Improper prestress force	6

Internationally, the most prevalent failure causes resulting in concrete crosstie and elastic fastening system deficiencies are fastening system damage, poor material quality or behavior, and manufacturing flaws. The least prevalent causes are poor bonding of concrete to prestress, deficient concrete strength, and improper prestress force. The low prevalence of these responses can perhaps be attributed to the predominance of the carousel manufacturing process. In North America, RSD was the most prevalent failure cause resulting in deficiencies, followed by fastening system damage and poor bonding of concrete to the prestress.

Figure 2.2 communicates the most important concrete crosstie and elastic fastening system topics of research from most to least critical as expressed by the international and North American responses.

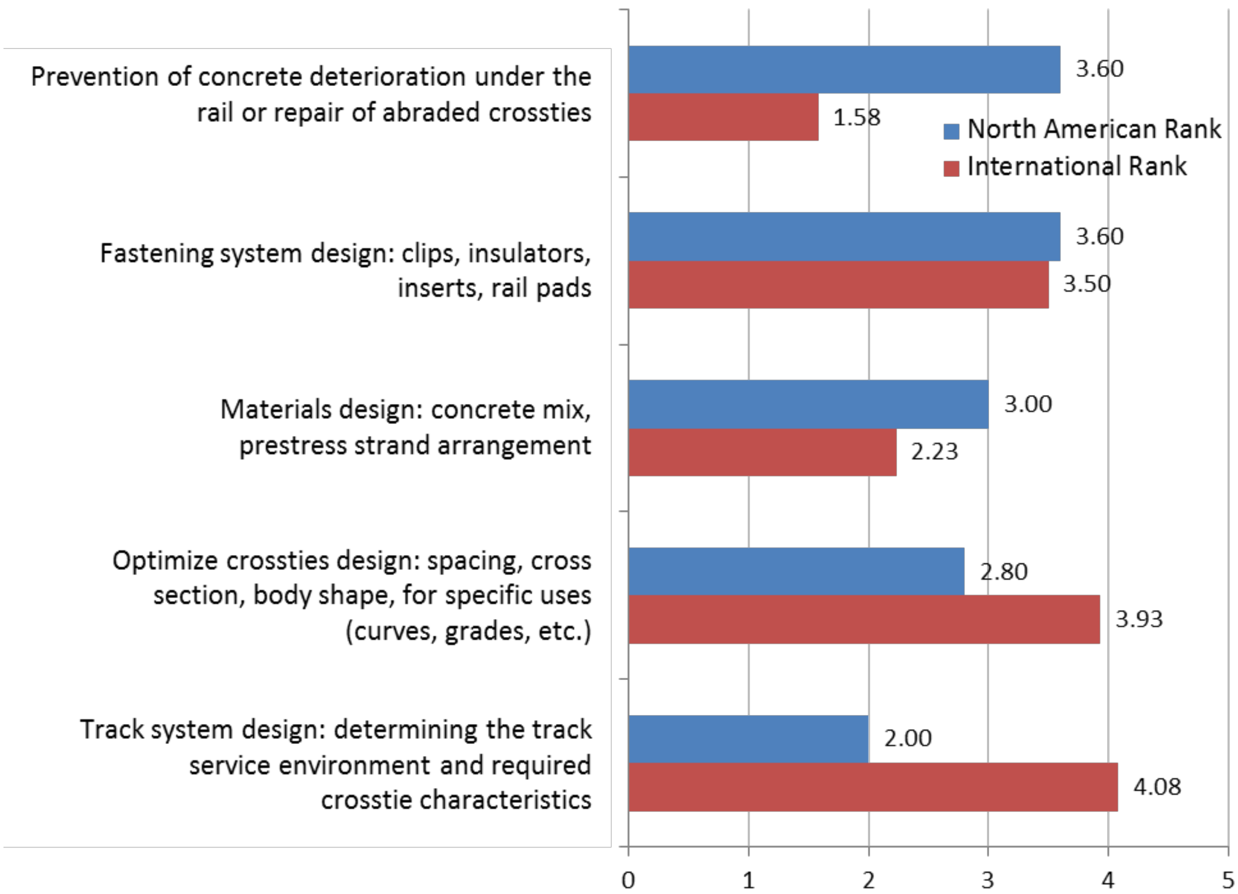

Figure 2.2 The most important concrete crosstie and elastic fastening system research topics; ranked from 1 to 5, with 5 being the most important

Interestingly, the international respondents indicated a reversed priority in research compared with the North American respondents. While the international respondents indicated track system design and crosstie design optimization as being the most important research topics, the North American respondents placed a high priority on RSD prevention and fastening system design. These North American research priorities are aligned with the current research thrusts at UIUC, and adjustments were made to ensure these thrusts remain consistent with the research needs identified in the International Survey. As a result, there are several projects being conducted concurrently with the FRA Tie and Fastener BAA related to mitigating RSD, and one of the primary objectives of the UIUC Tie and Fastener Research Program is to determine better mechanistic design recommendations for the crosstie and elastic fastening systems.

Table 2.3 provides a summary of the total responses while comparing the international and North American responses.

Table 2.3 Summary of Responses to International Concrete Crosstie and Fastening System Survey

	International Responses	North American Responses
Participant Demographics		
Total number of responses	18	10
Infrastructure owner, operator, or maintainer	5	4
Academic, industry, or institutional researcher	10	2
Concrete crosstie manufacturer	3	4
Loading Environment		
Average maximum freight axle load*	29.5 tons (26.8 tonnes)	39.1 tons (35.4 tonnes)
Average maximum passenger axle load*†	21.6 tons (19.6 tonnes)	29.1 tons (26.4 tonnes)
Average annual tonnage (per track)	38.7 million gross tons (35.1 million gross tonnes)	100.0 million gross tons (90.8 million gross tonnes)
Fastening system manufacturers	Vossloh, NABLA, JIS, Pandrol, Railtech	Pandrol, Vossloh, Unit Rail/Amsted RPS**
Concrete crosstie manufacturers	Austrak, SATEBA, RAIL.ONE, KNR, Parma, Luja, SSL, BK.International, Taemyung, Samsung, IS Dongseo, Sampyo	CXT, Koppers (KSA), Rocla, GIC**, ITISA**, Voestalpine Nortrak**
Average concrete crosstie design axle load	27.6 tons (25.0 tonnes)	37.4 tons (33.9 tonnes)
Average tangent crosstie spacing	24.2 inches (61.4 centimeters)	24.0 inches (61.0 centimeters)
Average concrete crosstie and fastening system years of use	48.4	30.0
Trends in Crosstie and Fastener Performance		
Average concrete crosstie design life (years)	35.0	41.7
Abrasion plate or frame	No	Yes
Commonly failed components	Screw, clip	Pad, rail seat
Rail seat deterioration	No	Yes
Focus of research	Loading, testing, design	Life cycle cost reduction
Average minimum allowable concrete strength at transfer	6500 psi (44.8 MPa)	4700 psi (32.4 MPa)
Average 28-day concrete compressive strength	8700 psi (60.0 MPa)	8250 psi (56.9 MPa)
Concrete crosstie manufacturing process	Carousel, long line**	Long line

*Interpreted from responses due to discrepancies in axle or wheel loads
**Added by report authors for completeness
†Light rail response excluded

To better understand the complex loading conditions within the concrete crosstie and elastic fastening system, it is important to understand what types of loads are being applied to that system. The maximum freight static axle load within the responses was 44.1 tons and the average maximum freight static axle load is 32.3 tons. Internationally and domestically, the average maximum freight static axle load exceeds the design axle load based on responses from the concrete crosstie manufacturers. To include dynamic considerations in the loading environment, impact factors must also be applied to the static axle loads, ranging from 130% to 300% (with most responses around 150 - 200%). While these averages provide some clarity in the loading environment experienced internationally, they do not sufficiently represent the full load spectrum. This concept will be revisited in Chapters 4 and 5.

As expected, the load and tonnage values are, on average, substantially higher in North America than in the remainder of the world, according to the respondents. Also, the trends in commonly failed components and use of an abrasion frame in North America coincide with the prevalence of RSD, as shown previously in Figure 2.1 and Table 2.2.

Another substantial finding displayed in Table 2.3 is the disparity in average minimum allowable concrete strength at transfer of prestress in the concrete crosstie. The concrete strength at transfer according to the North American respondents was only 72% of that reported by the international respondents, on average. This discrepancy is almost removed, however, once the 28-day compressive strength is recorded, as the North American 28-day strength is, on average, 95% of that internationally. Perhaps the difference in strength at prestress transfer is associated with the prevalent manufacturing processes (often carousel internationally and long line in North America).

2.5.3 Fastening System Manufacturer Survey

Because it was unlikely that the online survey would have been applicable to their unique global positions within the railway industry, the fastening system manufacturers were distributed a separate set of questions on an individual company basis. This list of questions was supplemented by personal conversations to discuss the current landscape of elastic fastening systems around the world and how their organizations contribute to that landscape. Due to the proprietary nature of the fastening system manufacturer responses, most of the results have not been included in this report. However, a few trends in the responses have been included.

For instance, in designing the fastening system, the following parameters are generally considered by the manufacturers: tonnage, daily train volume, velocity of trains, static loads, dynamic loads, the ability of the pad to evenly attenuate load to the rail seat, abrasion of the concrete rail seat by the pad or abrasion plate, and the curve radius. It is interesting to compare these considerations with those found in Chapter 30 of the AREMA Recommended Practices (American Railway Engineering and Maintenance-of-Way Association, 2012) for the concrete crossties themselves, which include tonnage, train speed, static loads (with impact factor), crosstie spacing, and crosstie length.

There were also noteworthy responses to the average life of the fastening systems. Responses varied from the life of the crosstie to the life of the rail, with the pad performing the most reliably of all the fastening system components. Shortcomings are most commonly seen in the insulator materials, while most failures occur in demanding operating environments with heavy curvature and steep grades.

2.6 Conclusion

There are several important conclusions that can be made as a result of this survey. First, the manufacturing process differences between the North American and international respondents may be the cause of substantially different trends in requirements and performance of concrete crossties. There may be some testing that could be conducted to better determine the correlation between these trends. The results also indicate that the most critical failure concerns in North America are related to wear or fatigue on the rail seat, rail pad, or shoulder, while more critical failure concerns internationally are tamping damage, cracking from dynamic loads, and shoulder wear. Finally, the design considerations of the fastening system manufacturers can be applicable to the design of concrete crossties and the system as a whole. The fastening system manufacturers indicated that component and system interaction plays a large role in their design, and this concept should be considered in the development of mechanistic design recommendations for concrete crossties and elastic fastening systems.

CHAPTER 3: CONSIDERATIONS FOR MECHANISTIC DESIGN OF CONCRETE CROSSTIES AND ELASTIC FASTENING SYSTEMS IN NORTH AMERICA[2]

3.1 Introduction

Historically, the North American concrete crosstie and elastic fastening system has been designed through a process that is generally based on practical experience, without a clear understanding of failure mechanisms, their causes, and the loading environment. This design methodology has led to performance challenges and service failures that cannot be adequately explained or predicted. Without a clear framework for the design of concrete crossties and elastic fastening systems, inefficiencies in component design and manufacturing may exist, negatively impacting the economics of concrete crossties and elastic fastening systems. Improvements in the design of these systems will provide a more robust railway superstructure, where the loading environment is more fully considered, failures are reduced, and the possibility of predicting performance metrics (e.g. wear rates) exists.

The North American loading environment differs from much of the rest of the world (see Table 2.3), due to the prominence of rail freight transport and sharing of infrastructure between freight and passenger traffic. This chapter will investigate the particular loading conditions found in North America and draw comparisons between the varied international design considerations that are based on a variety of loading and operational environments.

3.2 Current Recommended Design Practices

Internationally, there are many unique design methodologies for the manufacture of concrete crossties and elastic fastening systems. Many countries have their own version of design standards or recommended practices that railways and manufacturers follow to varying degrees. This chapter will briefly discuss the similarities and differences in design methodologies found in North America, Europe, Australia, and Japan. Additional information on design requirements can be gained through a review of the International Concrete Crosstie and Fastening System Survey, described in Chapter 2.

[2] Much of Chapter 3 was originally published in the Proceedings of the 2013 International Heavy Haul Association Conference in New Delhi, India (Van Dyk et al. 2013)

The American Railway Engineering and Maintenance-of-way Association's (AREMA) Manual for Railway Engineering is the primary source of guidance for the design and construction of North American rail infrastructure. It is a set of recommended practices, and is typically modified by individual railways to meet their specific loading or performance objectives (American Railway Engineering and Maintenance-of-Way Association 2012). Chapter 30 of the Manual for Railway Engineering provides guidance for crossties, and Part 4 of that chapter focuses on concrete crossties. While this section of the AREMA manual offers helpful information for railways and crosstie manufacturers, there are opportunities for improvement, particularly in terms of the crosstie design process (hereafter referred to as the "AREMA Method").

One opportunity for improvement of AREMA Chapter 30 is the consideration of component interactions and system performance. In the 2012 International Concrete Crosstie and Fastening System Survey conducted by UIUC (Chapter 2), fastening system manufacturers indicated that component and system interaction plays a large role in fastener design. This concept should be included in the development of improved design recommendations for concrete crossties and elastic fastening systems.

Two of the most significant design parameters used in the AREMA Method for determining concrete crosstie geometric and strength characteristics are allowable ballast pressure and flexural performance. In determining the allowable ballast pressure, the AREMA Method considers crosstie spacing (leading to the determination of a load distribution factor), wheel load, an assumed impact factor, and crosstie bearing area. Another portion of the AREMA Method for concrete crosstie design contains the flexural performance requirements. These requirements consider crosstie length, crosstie spacing, speed, and tonnage to determine the positive and negative design bending moments at the center of the crosstie and at the rail seat. Some consideration was given to impact factors and axle loads in the fabrication of the method, but they were developed using particular operating and track characteristics (including uniform ballast support) and applied equally to all cases related to the flexural design process (McQueen 2010). Therefore, the flexural design of a concrete crosstie as found in AREMA Chapter 30 does not consider many important design criteria, such as track geometry (e.g. curvature and grade), design life, or impact factors and axle loads that reflect the intended loading environment.

Because it is typically the primary design criteria for concrete crossties, the authors have reviewed the bending moment design methodologies in multiple standards and recommended practices. Within each methodology, different design principles are considered and used.

The European Standard offers recommendations for the design of concrete crossties, and, like the AREMA Method, its primary focus is the design bending moment. However, EN 13230 states that the specific design method is the responsibility of the purchaser, considering static and dynamic wheel loads, design and maintenance of the track (including longitudinal distribution of wheel loads), climatic conditions, magnitude of prestressing force, strength of concrete, and particular, non-standard designs (European Committee for Standardization 2009).

The Australian Standard calculates positive and negative rail seat and center bending moments using crosstie spacing, static wheel load, track modulus, rail modulus, rail second moment of area, quasi-static and dynamic design load factor, crosstie length, gauge, and support conditions (Standards Australia International 2003). An intermediate step to this process incorporates Talbot's method for determining rail seat loads (Hay 1982). The standard also explicitly states that crosstie sections need not be checked for stresses other than flexural stresses (Standards Australia International 2003).

The Japanese Industrial Standard (JIS) simply provides "bending forces" that must be exceeded during testing of concrete crossties (Japanese Standards Association 1997). The design methodology is not explicitly provided in the JIS, and is therefore determined by the manufacturer, as long as it meets the performance criteria as stated in the JIS.

After reviewing the above international design methodologies, it is evident that the concrete crosstie design process is not uniform throughout the international railway community. There are many criteria to be considered from design recommendations and best practices worldwide. These principles can be applied to the development of an approach that is centered on mechanics and materials properties to govern the design of concrete crossties and elastic fastening systems in North America. However, the operating environment in North America, which is often different than that found elsewhere in the world, must be better understood before mechanistic design recommendations can fully be developed and placed into practice.

3.3 Principles of Mechanistic Design

The mechanistic design process is one derived from analytical and scientific principles, considering field loading conditions and performance requirements. Some form of mechanistic design has been used in other disciplines, such as the design of rigid and flexible highway pavements using particular input values, performance analyses, and alternative evaluations (ARA, Inc. 2004).

Historically, North American concrete crossties and elastic fastening systems have been designed through a design process that does not include all of the critical variables relating to actual field loading conditions. A lack of understanding regarding the complex loading conditions of the system has led to a design methodology driven by production and installation economics, where very high priority is placed on manufacturing and installation efficiency. Oftentimes, this process is not directly based on actual performance of the crosstie and fastening system or a thorough understanding of the demands on each component.

Therefore, UIUC is developing a mechanistic design process that uses the existing loading environment on and between the crosstie and fastening system components. This exercise will create an improved understanding of failure causes and their effects on performance. System and component design would typically be directed toward a specific failure mode (often grouped into one of three categories; support, stability, or isolation failure (Zeman 2010)), creating predictable wear and fatigue rates and leading to repair cycles that coincide with other planned maintenance intervals. This improved design procedure will increase production and operational efficiency while reducing unscheduled maintenance and track outages.

3.3.1 Shared Use Loading Environment in North America

The railway operating environment in North America is different than much of the rest of the world. As enthusiasm for higher-speed intercity passenger service grows, some systems are developing that require passenger and freight traffic to share the same infrastructure. Shared railway infrastructure provides an effective method for providing an incremental approach to higher-speed passenger transportation, and reduces the first cost associated with opening a new system. One of the many challenges facing shared use infrastructure is the design and performance of critical components such as the crosstie and fastening system. To better understand loads applied to the infrastructure, UIUC has acquired Wheel Impact Load Detector (WILD) data from sites throughout the United States from both Amtrak's Northeast Corridor, (a shared use corridor in operation for many decades), and the Union Pacific Railroad.

WILD sites are typically constructed on well-maintained tangent track with concrete crossties, premium ballast, and well compacted subgrade (possibly with hot mix asphalt underlayment) to reduce sources of load variation within the track structure. Although loads experienced elsewhere on the network will vary and may have a higher magnitude due to track geometry deviations, these data still provide insight to the varied loading landscape at representative sites throughout North America. Specific loading properties such as peak vertical load, peak lateral load, impact factor, and speed are analyzed by creating various distributions of these properties and determining relationships between them. An example of this type of distribution is shown in Figure 3.1.

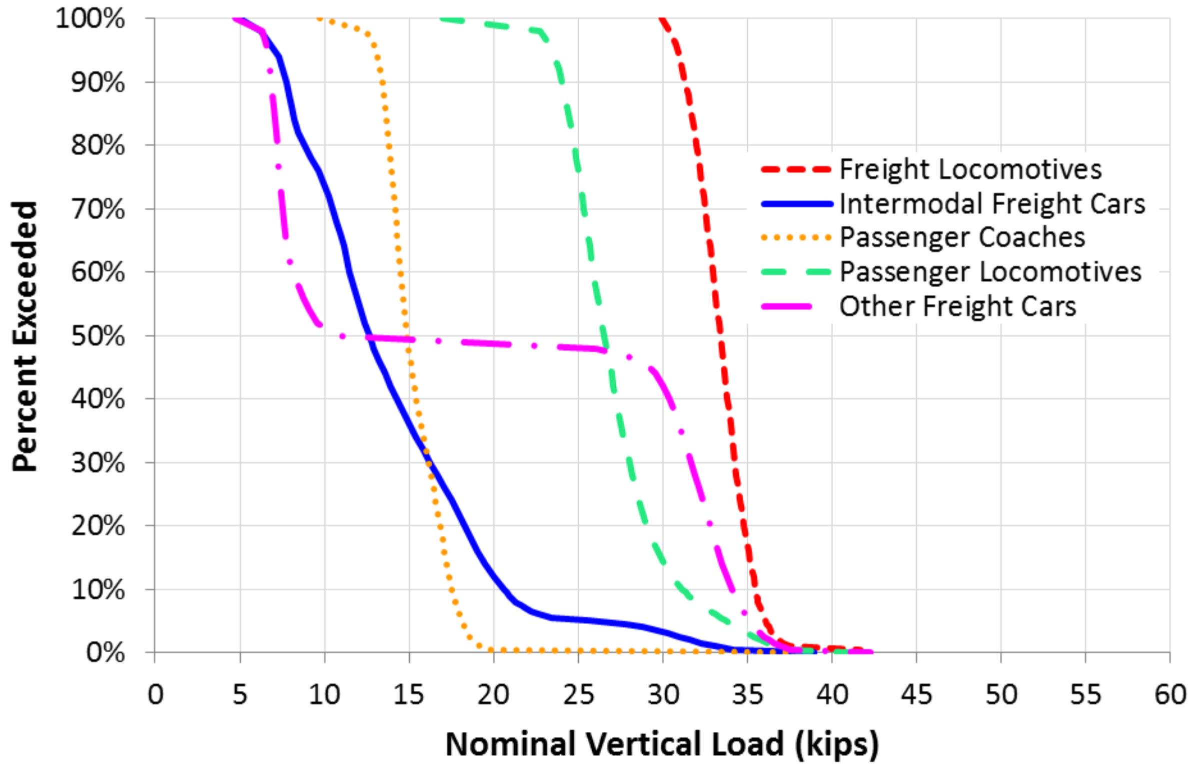

Figure 3.1 Percent exceeding particular nominal vertical loads on Amtrak at Edgewood, Maryland (WILD data from November 2010)

As Figure 3.1 shows, at Amtrak's Edgewood, MD WILD site, locomotives, freight cars, and passenger coaches all impart different magnitudes of vertical load into the track structure. Once the loading spectrum is adequately determined, one must decide how to effectively design the system and its components accordingly. The relationship between extreme loading events (e.g. wheel impact loads) and failure mechanisms is not well-defined, so it is difficult to sufficiently determine the required robustness of design. Probabilistic considerations must be made throughout the design process, reflecting safety, financial, and capacity decisions. The disparity in the magnitude of loads between passenger and freight traffic and their respective weighted traffic volumes must also be addressed in designing for specific loading environments.

Results from the 2012 UIUC International Concrete Crosstie and Fastening System Survey, described in Chapter 2, provide a comparison of the North American and international loading environments and are summarized in Table 3.1. According to both the international and North American responses, the average maximum freight static axle load exceeds the design axle load based on responses from the concrete crosstie manufacturers. The load and tonnage values are, on average, substantially higher in North America than in the remainder of the world, according to the respondents (Table 3.1).

Table 3.1 Loading Environment Summary from the 2012 International Concrete Crosstie and Fastening System Survey

	International Responses	North American Responses
Average maximum freight axle load*	29.5 tons (26.8 tonnes)	39.1 tons (35.4 tonnes)
Average maximum passenger axle load*†	21.6 tons (19.6 tonnes)	29.1 tons (26.4 tonnes)
Average concrete crosstie design axle load	27.6 tons (25.0 tonnes)	37.4 tons (33.9 tonnes)
Average annual tonnage (per track)	38.7 million gross tons (35.1 million gross tonnes)	100.0 million gross tons (90.8 million gross tonnes)

*Interpreted from responses due to discrepancies in axle or wheel loads
†Light rail response excluded

Both the WILD data and survey results provide a better understanding of the loads imparted into the superstructure, but this understanding is not sufficient for the design of concrete crossties and elastic fastening systems. The load's attenuation and progression through the track provides information critical to the design of the superstructure components.

3.3.2 Qualitative Establishment of Load Path

At their core, mechanistic design practices use actual loading data to develop a design that functions adequately under the expected loading conditions. To better determine the demands on each component, an analysis of the static load path was conducted at UIUC. This analysis underwent several iterations with increasingly detailed assumptions. This static analysis of interface loads and component deflections, described in the following sections, helped to establish the locations for load transfer that may require additional analysis.

Given a particular input loading condition and appropriate simplifying assumptions, the magnitude of forces at each interface can be determined. UIUC is developing software (I-TRACK) that accepts particular input parameters, such as material and geometrical component properties, and produces forces at interfaces and component deflections. Therefore, the spectra of loads, such as those shown in Figure 3.1, can be traced throughout the remainder of the fastening system (and the crosstie, ballast, and subgrade), providing estimates of the magnitudes of forces that should be measured at each interface given a particular traffic type.

In addition to this initial analysis, the effect of accelerating wheel loads and clamping force on longitudinal forces must also be considered in a comprehensive exploration. Because many simplifying assumptions were used to complete this initial investigation, its results must be viewed as an estimate, providing feasible values to be compared with other load quantification efforts. To evaluate the loads within the system more accurately, lab and field instrumentation and more sophisticated analyses, such as finite element analysis (FEA) techniques, must be employed (Section 3.3.3).

3.3.2.1 Rigid Body Analysis

The first attempt at mapping the forces throughout the fastening system required several simplifying assumptions, as follows:

- Neglect system and component deflections (i.e. components idealized as rigid bodies)
- Neglect self-weight of each component
- Fastening system consists of a single pad, two insulators, and two elastic clips (Amsted RPS U2000, similar to the Pandrol Safelok I)
- The clip is driven and all fastening system components are correctly installed
- Axis orientation is as follows:
    - Z-axis is normal to the inclination of the rail seat
    - X-axis is parallel to the inclination of the rail seat
    - Y-axis is parallel to the longitudinal direction of the rail
- Neglect tangential forces; where they are necessary, substitute moments

To simplify the procedure and allow for a better understanding of individual loading, load path maps were created by separating into three distinct load cases:

a. Clamping force only

b. Vertical external load only

c. Horizontal external load only

d. Summation of loads due to cases (a), (b), and (c)

Case (a) includes the forces within the fastening system due to the clamping force exerted by the driven clip (Figure 3.2). Case (b) includes the forces within the fastening system that can be attributed to a purely vertical external load applied to the rail head (Figure 3.3). Case (c) includes the forces within the fastening system that exist due to a purely horizontal load applied to the rail head (Figure 3.4).

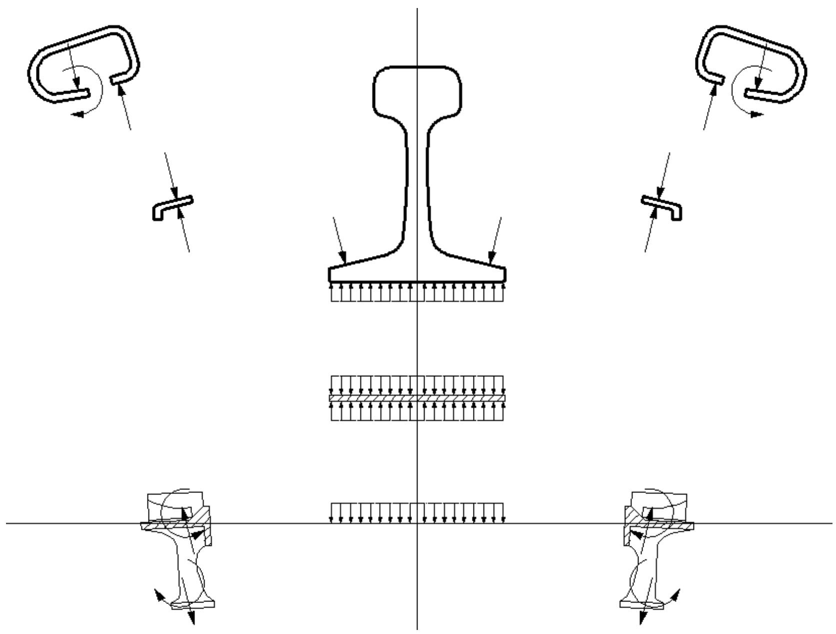

Figure 3.2 Concrete crosstie fastening system load path map: case (a), forces due to clamping force (rigid bodies)

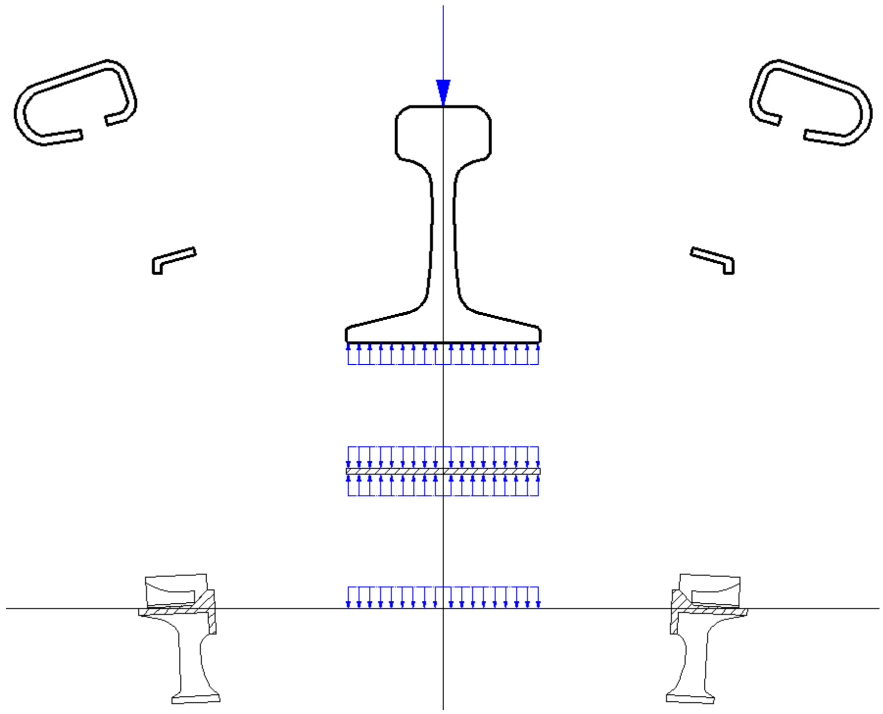

Figure 3.3 Concrete crosstie fastening system load path map: case (b), forces due to vertical external load (rigid bodies)

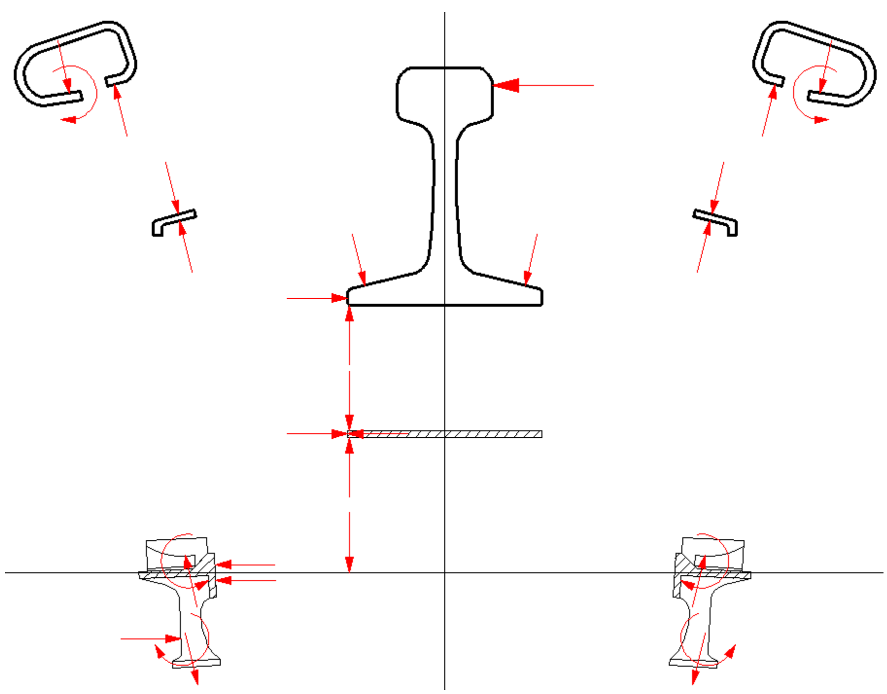

Figure 3.4 Concrete crosstie fastening system load path map: case (c), forces due to lateral external load (rigid bodies)

To complete the load path map and component free body diagrams, all three load cases were combined using the concept of superposition (case (d)) (Figure 3.5). This combination adequately models the field conditions of the fastening system. Where forces from multiple load cases exist at the same location, they were represented by superimposed forces of different colors, creating a clear representation of all forces acting externally on each component (Figure 3.5).

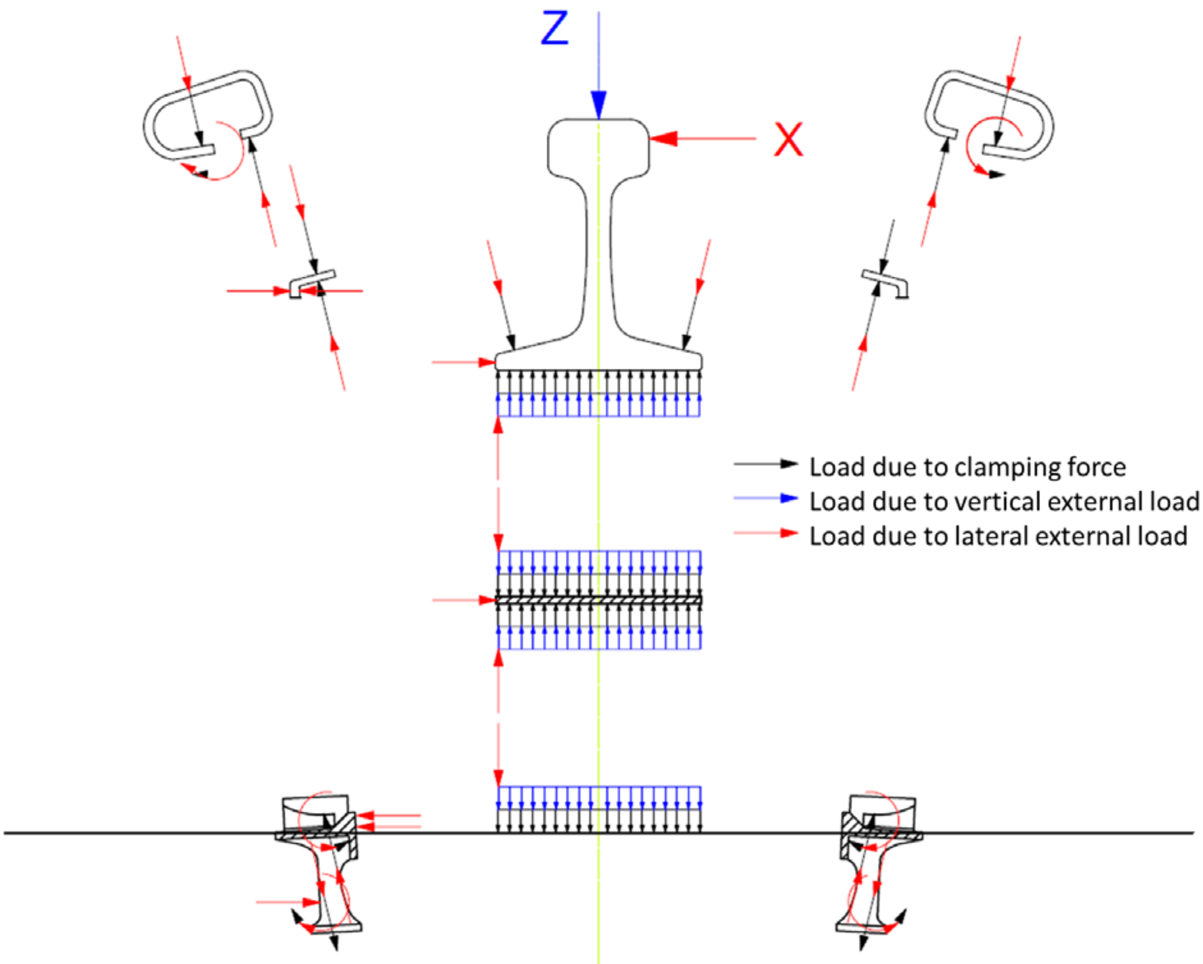

Figure 3.5 Concrete crosstie fastening system load path map and component free body diagram, case (d) (rigid bodies)

3.3.2.2 Deformable Body Analysis

The second iteration of this analysis included the more realistic condition where the components are deformable. Therefore, the same assumptions were included as above, with the following changes:

- Include system and component deflections (i.e. components no longer idealized as rigid bodies)

- Incorporate relative component stiffness into analysis
- The base of the rail is infinitely stiff, thus producing an idealized pressure distribution at the rail seat
- The surface bond between the cast-in shoulder and concrete is idealized as a single point load to create a balanced moment condition
- Neglect tangential forces

Using the same cases (a-d) as above, load path maps were developed for the Amsted RPS U2000 (Pandrol Safelok I type) elastic fastening system (Figures 3.6 – 3.9).

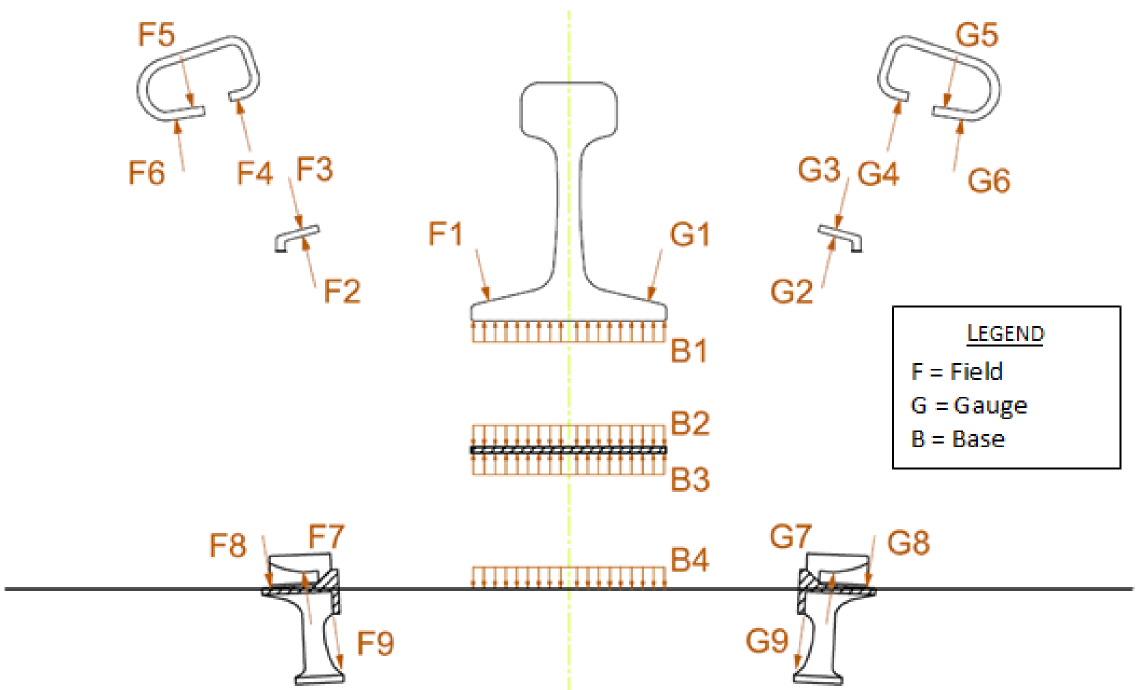

Figure 3.6 Concrete crosstie fastening system load path map: case (a), forces due to clamping force (deformable bodies)

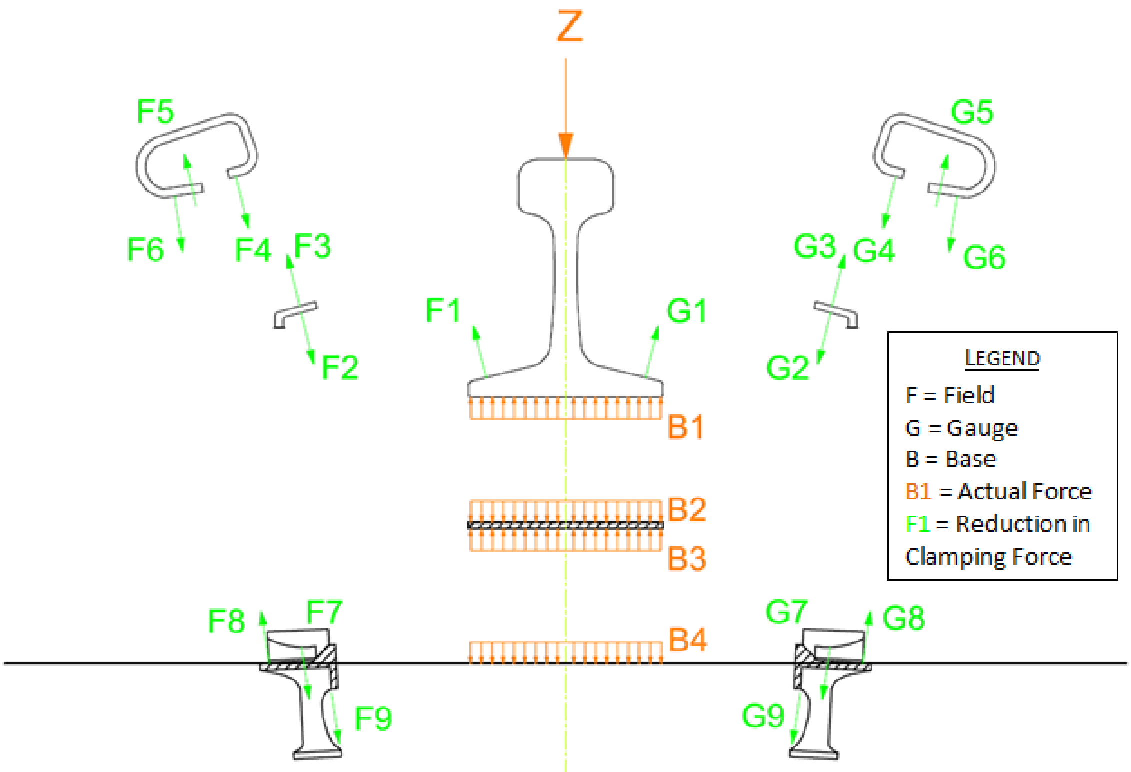

Figure 3.7 Concrete crosstie fastening system load path map: case (b), forces due to vertical external load (deformable bodies)

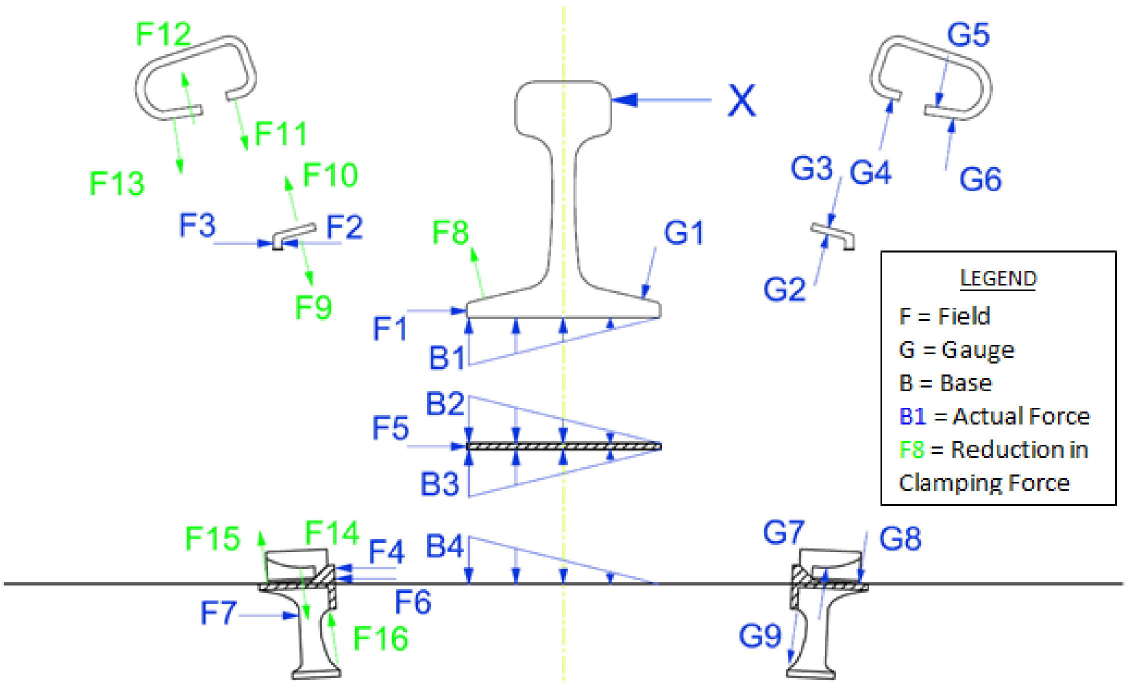

Figure 3.8 Concrete crosstie fastening system load path map: case (c), forces due to lateral external load (deformable bodies)

After a review of the available literature, discussions involving thought experiments, and simple finite element analyses, it was determined that the rail's center of rotation was much closer to the center of the rail base. Therefore the force distribution between the rail and pad would not extend across the entire rail base in the pure lateral external load case (c). A revised load path map was subsequently developed reflecting this improved understanding (Figure 3.9).

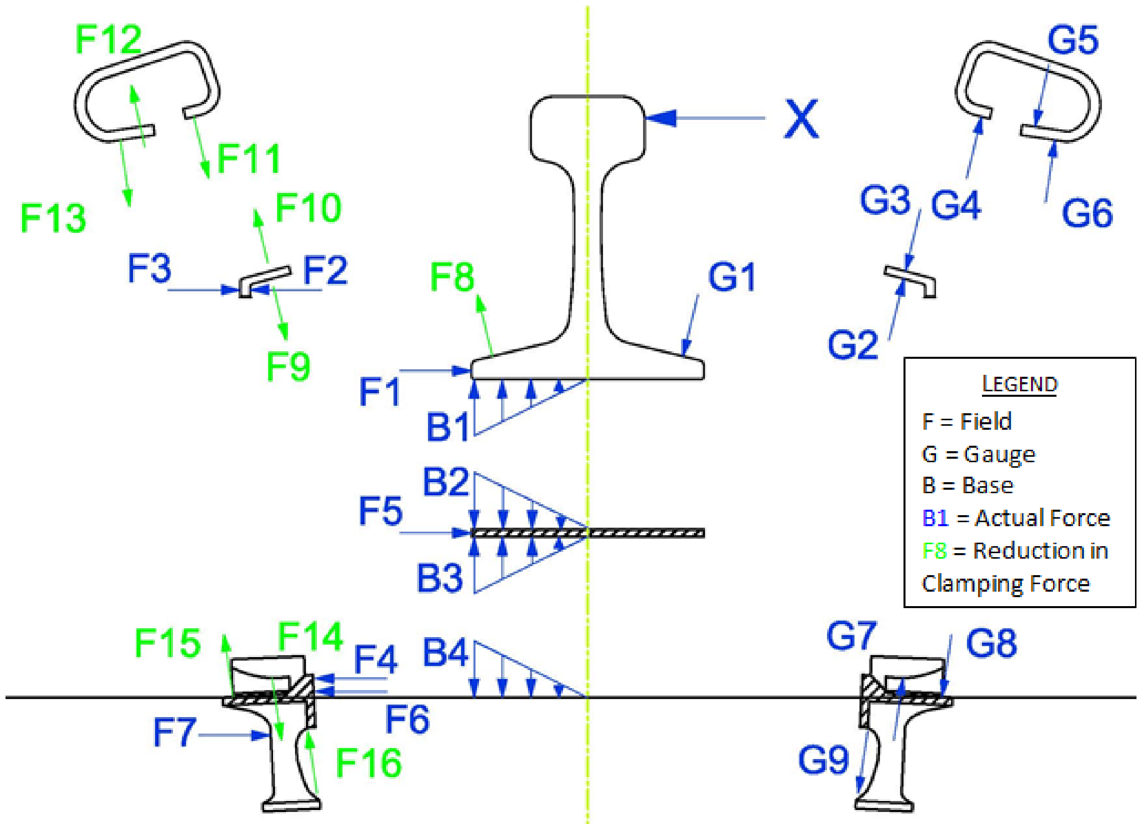

Figure 3.9 Revised concrete crosstie fastening system load path map: case (c), forces due to lateral external load (deformable bodies)

To complete the load path map and component free body diagrams, all three load cases were again combined using the concept of superposition. Where forces from multiple load cases exist at the same location, they were combined to concisely represent all forces acting externally on each component. An improved naming convention was also used to more clearly represent the components (Figure 3.10).

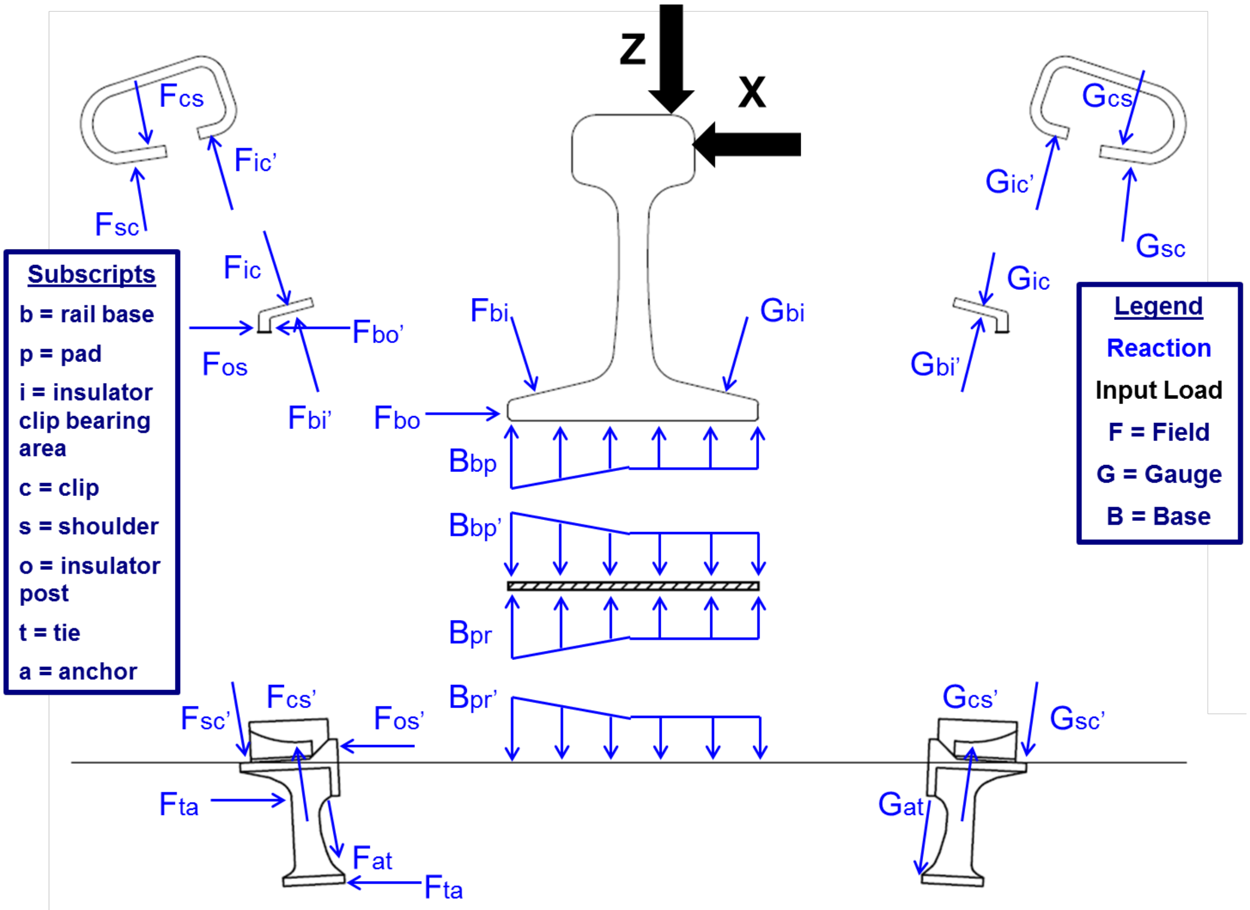

Figure 3.10 Concrete crosstie fastening system load path map and component free body diagram, case (d) (deformable bodies)

3.3.3 Laboratory Experimentation, Field Instrumentation, and Analytical Modeling

After identifying locations where the load is transferred throughout the system, it is necessary to try to accurately quantify the loads that were qualitatively derived. This quantification process defines the demands on each component, focusing primarily on determining the magnitude of forces that are transferred at component interfaces. Laboratory experimentation, field instrumentation, and analytical modeling are tools used to quantify the loading conditions and displacements at each interface between components.

Both laboratory and field instrumentation provide quantitative information regarding the load path through the concrete crosstie and elastic fastening system. Using known input loads from full-scale testing in the laboratory and revenue service testing in the field, UIUC has developed a method for determining loads applied to the components within the system and their associated deflections (Grasse 2013). To correlate the interfacial loads with wheel loads applied at the wheel/rail interface, substantial instrumentation is used on the rail as well. In all, the magnitudes of the following measurements are acquired in the laboratory and field settings: vertical wheel load, lateral wheel load, longitudinal wheel load, vertical rail strain, rail base bending stress, vertical rail displacement, lateral rail displacement, rail rotation, lateral pad assembly displacement, longitudinal pad assembly displacement, longitudinal crosstie displacement, global vertical displacement, global lateral displacement, internal crosstie strain, external crosstie (surface) strain, vertical rail seat load, rail seat stress distribution, insulator post stress, lateral force entering the shoulder, fastening clip stress, and pad temperature. The analysis of these values provides a substantially improved understanding of the behavior of the concrete crosstie and elastic fastening system as a whole.

In addition to the instrumentation performed on the physical system, extensive three-dimensional (3D) analytical methods are also employed. Using the qualitative free body diagrams as shown in Figure 3.10 as a framework, as well as basic statics principles, a fundamental analysis was performed to determine estimated loads and deflections of the components. Simplified two-dimensional (2D) finite element models were created to confirm the basic analysis and provide further guidance to the forces present within the system (Chen, Shin & Andrawes 2013). In parallel with both the instrumentation and basic analysis, a comprehensive finite element model was created incorporating the geometry and materials of each component and its interaction with those surrounding it (Chen, Shin & Andrawes 2013). This tool can model different loading scenarios, ultimately including dynamic loads, and provide valuable insight into the component response and interdependencies. Parametric analyses were performed, guiding our understanding of component properties and how they relate to the performance within the expected loading regime. Once validated, the model will ultimately be the primary tool for running iterations that will facilitate the development of mechanistic design practices.

3.3.4 Design Process

After gaining an improved understanding of the loading environment, one must look at the current geometry and material properties of the components and evaluate whether or not those properties are appropriate for the existing and expected loading environment. If not, alternative component geometries or materials that can better endure the existing loading demands should be pursued.

The next step in the design process is to relate the loading conditions to specific failure modes. This is done by identifying certain types of failure that occur specifically because of the loading demands on that particular component. Taking advantage of the modeling techniques, innovative designs can be developed and tested using the instrumentation plan already in place. Existing geometry and materials can continually be improved, and some completely novel component designs could be developed. Ultimately, this process will lead to improved mechanistic design practices. This set of recommendations will be based on both theoretical and empirical relationships, leading to a more thorough understanding of the behavior and performance of each component.

3.4 Conclusions

The original development of the AREMA recommended practices did not fully consider the complex loading conditions found within today's concrete crossties and elastic fastening systems in North America. An improved understanding of the existing loading environment will provide greater insight into failure mechanisms. The cause of these failure modes can be addressed by improvements to design recommendations based on the science of those mechanisms. Ultimately, the mechanistic process of design will lead to improved performance of concrete crossties and elastic fastening systems, increased safety, and decreased life cycle costs.

CHAPTER 4: LOAD CHARACTERIZATION TECHNIQUES AND OVERVIEW OF LOADING ENVIRONMENT IN NORTH AMERICA[3]

4.1 Introduction

As discussed in Chapter 3, elements of the track superstructure in North America have historically been designed through a process that is generally based on practical experience, without a complete understanding of the loading environment causing particular failure mechanisms. Improvements in the design process for track superstructure components may result in a more robust track structure if the loading environment can be adequately characterized.

The North American operating environment differs from that found throughout much of the rest of the world due to the prominence of heavy axle load rail freight transportation and shared infrastructure between heavy axle load freight and intercity passenger rail traffic. One of the challenges created by this operating environment is the design of critical infrastructure components under a widely varied loading spectrum.

To best determine how to describe the loads entering the track structure, one must explore possible causes of variation. This chapter will use data, primarily from wheel impact load detectors (WILD), to identify sources of variation in the loading regime entering the track structure and test several hypotheses aimed at understanding trends between some of the most critical parameters. These hypotheses are that (a) the static load is the most reliable indicator of wheel load, (b) increased speed causes increased wheel loads, (c) conditions prevalent in the winter months result in higher wheel loads, and (d) site-based traffic composition has a substantial influence on the distribution of loads at the wheel-rail interface. Instrumented wheel set (IWS) data will be used to explore the effect of curvature and cant deficiency on wheel load magnitudes. More thorough understanding of these relationships will lead to improved design effectiveness of critical infrastructure components.

[3] Much of Chapter 4 was originally published in the Proceedings of the 2013 Joint Rail Conference (JRC) in Knoxville, Tennessee, USA (Van Dyk et al. 2013b)

4.2 Methodologies and Measurement Technologies

There are several load quantification technologies, systems, and instrumentation strategies available to the rail industry for quantifying the performance of vehicles and track. Specifically, instrumented wheel sets (IWS), truck performance detectors (TPD), and wheel impact load detectors (WILD) monitor forces at the wheel-rail interface. These systems are used to monitor rolling stock performance and assess wheel and vehicle health, producing efficiencies in both predictive and reactive maintenance strategies. However, they can also be used by railway infrastructure engineers to provide insight into the magnitude and distribution of loads entering the track structure. A clear understanding of this loading spectrum provides a foundation for the analysis and design of critical infrastructure components.

4.2.1 Instrumented Wheel Set (IWS)

The IWS is a wheel set that is instrumented with strain gauges on the axle and wheels (Figure 4.1). It can be deployed on any type of vehicle and provides information related to vertical, lateral, and tangential forces created by the wheel set, as well as the contact patch location on the head of the rail. The IWS measures numerous data channels (Table 4.1) at high frequencies (300 Hz) which, through the use of GPS referencing, can be combined with other recorded track data (e.g. track geometry, curvature, grade, type of track structure, track stiffness). While the IWS data is primarily used to evaluate rolling stock component and system performance, it can also be used to determine the magnitude of the forces being imparted to the track. In the future, UIUC will further utilize IWS data from the Association of American Railroads (AAR) and TTX Company to provide insight into the effects of these track parameters on forces experienced at the wheel-rail interface.

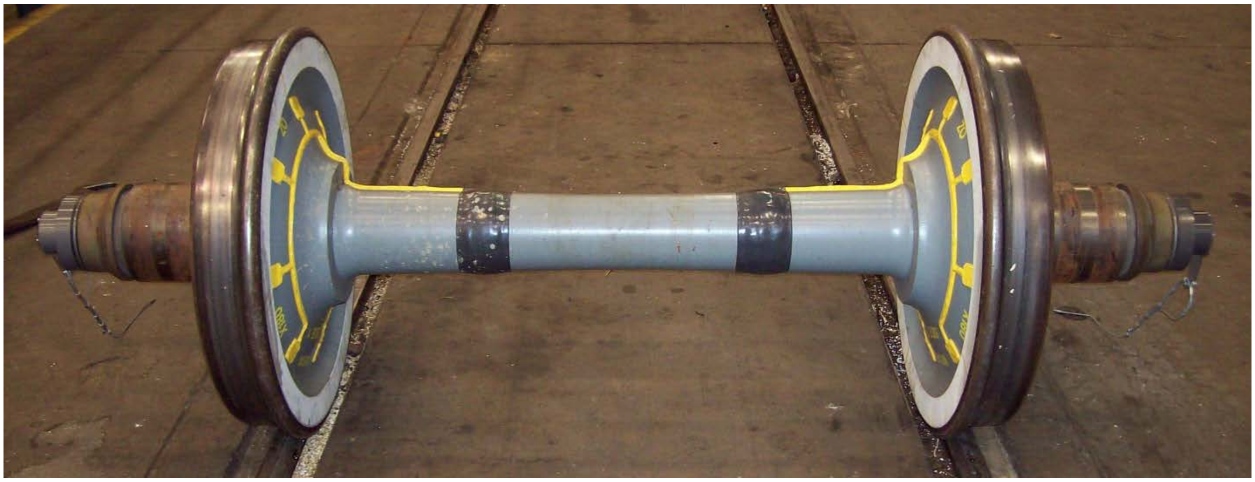

Figure 4.1 An instrumented wheel set used for research and development (TTX Company)

Table 4.1 Information produced by a typical IWS

Data Type	Number of Channels	Description
Axle Torque	2	Axle torque
Carbody Acceleration	13	Acceleration of part of carbody
Contact Patch Location	4	Location of contact patch with respect to datum
Diagnostic of Measurement	8	Diagnoses other data channels
Reference	19	Provides reference for data (e.g., coordinates, time, distance, ALD location)
Truck Component Strain	38	Strain of particular component of truck
Wheel Load Calculation or Ratio	14	Calculation of wheel load or ratio of multiple wheel load measurements
Wheel Set Speed	2	Speed of wheel set
Wheel/Axle Strain	46	Strain at particular location on wheel or axle

4.2.2 Truck Performance Detector (TPD)

A TPD is a wayside device that utilizes strain gauges to measure vertical and lateral forces on the low and high rail at a field location that has a reverse curve separated by a short segment of tangent track. The TPD measures and records vehicle response through the curve to evaluate the curving performance of the truck and vehicle (Salient Systems, Inc. 2005). It also includes two circuits within the tangent section between the curves to measure vertical and lateral wheel-rail forces. Some versions of the detector include eight additional circuits in that section acting as a “weigh-in-motion” device. This type of device often stands alone and is used elsewhere on railway networks to provide information related to the load magnitude and load distribution of passing vehicles (Venekamp & Boom 2010). Figure 4.2 displays a general schematic and Table 4.2 shows the information provided by a typical TPD.

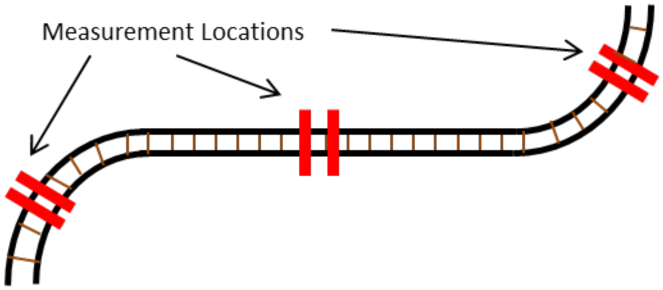

Figure 4.2 Schematic of typical TPD

Table 4.2 Information produced by a typical Progressive Rail Technologies TPD

Train-Specific Information	
Site name	
Date	
Time	
Total number of axles in train	
Total number of cars in train	
Train direction	
Average train speed (mph) through site	
Train type	
Maximum axle load (tons) within train	
Average axle load (tons) within train	
Average car weight (tons) within train	
Tonnage (tons) of entire train	
Vehicle-Specific Information	
Vehicle owner and number	
Gauge spreading index (GSI) (proprietary performance index)	
Vehicle type	
Data Channel	**Description**
V	Validity code for readings
TRAX	Absolute axle location in train
CRAX	Absolute axle location relative to car
AXLE	Truck axle designator and wheel indicator (L or R)
TRUCK	Truck designator for the car (A or B)
VLOAD	Vertical wheel load (kips)
LFORCE	Lateral wheel load (kips)
TSLV	Truck side lateral/vertical force ratio
SWLV	Single wheel lateral/vertical force ratio
ASLV	Axle sum lateral/vertical force ratio
AOA	Axle angle of attack (mrad)

4.2.3 Wheel Impact Load Detector (WILD)

A WILD consists of strain gauges mounted on the rail over a series of cribs that measure vertical rail strain to calculate wheel loads. A WILD site is over 50 feet in length, with the rail instrumented at various intervals to capture a single wheel's rotation five times, recording peak (impact) forces, as well as average forces (Canadian National Railway 2011) by collecting data at 25 kHz. Using an algorithm that analyzes variability along the site, these average, or nominal, forces are filtered from the peak loads to obtain an estimate of static wheel load. The peak wheel load is simply the highest recorded measurement from the strain gauges along the length of the detector. Additional information produced by the WILD is shown in Table 4.3. While the WILD has traditionally been used by infrastructure and rolling stock owners to detect and identify poorly-performing wheels, it has also been proven to be a practical mechanism for producing reliable wheel load data, according to a study performed by the AAR in which they reviewed the variation of measurements produced by the detector (Wiley & Elsaleiby 2007).

Table 4.3 Information produced by a typical Salient Systems WILD for each wheel passing over the WILD site

Data Channel	Description
Location	Location of WILD site
Track	Track number
Date	Date of train pass
Time	Time of train pass
Train Number	Reporting number of train passing
Reporting Marks	Vehicle owner
Service Type	Type of train (passenger or freight)
Car Type Desc	Type of vehicle
Car Number	Vehicle number
Direction	Direction of train movement
Rail	Indication of rail (E, W, or N, S) where measurement was taken
Speed	Speed (mph) of train at time of passing
Car Weight	Sum of nominal loads (kips) of all wheels within vehicle
Car Gross Tonnage	Sum of nominal loads (kips) of all non-locomotive wheels within train
Loco Gross Tonnage	Sum of nominal loads (kips) of all locomotive wheels within train
Gross Tonnage	Sum of nominal loads (kips) of all wheels within train
Car Count In Consist	Vehicle's position within train
Car Count	Number of non-locomotive vehicles within train
Loco Count	Number of locomotives within train
Car Axle Count In Consist	Number of non-locomotive axles within train
Loco Axle Count In Consist	Number of locomotive axles within train
Total Axle Count In Consist	Number of axles within train
Vehicle Direction	Direction of vehicle in train (A or B)
Axle Number	Axle designator within vehicle
Axle Count	Number of axles within vehicle
Axle Mass	Sum of nominal loads (kips) of both wheels within axle
Wheel Number	Wheel designator within axle (L or R)
Nominal Load	Average vertical wheel load (kips), calculated from sixteen sets of strain gauge readings; provides an estimation of static vertical wheel load
Peak Load	Maximum vertical wheel load (kips), selected from sixteen sets of strain gauge readings
Dynamic Load	Difference between peak load and nominal load (kips)
Ratio	Ratio of peak load to nominal load
Lateral Nominal Load	Average lateral wheel load (kips), calculated from sixteen sets of strain gauge readings
Lateral Peak Load	Maximum lateral wheel load (kips), selected from sixteen sets of strain gauge readings

Figure 4.3 WILD site on the Canadian National Railway (Canadian National Railway 2011)

WILD sites are constructed on tangent track with concrete crossties (Figure 4.3), typically with premium ballast, and well-compacted subgrade (possibly with hot mix asphalt underlayment) to reduce sources of load variation within the track structure due to track geometry and support condition irregularities. Although loads experienced in other locations on the network may have higher magnitudes due to track geometry and support deviations, these data still provide representative loading information for networks throughout North America (Van Dyk et al. 2013a).

Because WILDs are implemented to detect poorly-performing wheels and are, therefore, only located on tangent track where lateral to vertical load ratios (L/V) are typically less than 0.1, the information regarding lateral loads may not be as useful as compared to data collected on curved track. Therefore, much of the analysis shown in this chapter is derived from vertical loading data. Other measurement technologies may be useful for gathering loading data related to additional objectives, as shown in Table 4.4. It is the intent of the UIUC research team to further develop our understanding of lateral loads through the use of other technologies, such as the IWS and TPD.

Table 4.4 Comparison of load measurement technologies

	Measurement Technology			
Capabilities	**Instrumented Wheel Set (IWS)**	**Truck Performance Detector (TPD)**	**Wheel Impact Load Detector (WILD)**	**UIUC Instrumentation Plan**
Implementation location	Vehicle-mounted	Wayside	Wayside	Wayside
Continuous data with respect to	Vehicle	Track	Track	Track
Measures speed	Yes	Yes	Yes	No
Measures nominal vertical load	Yes	Yes	Yes	Yes
Measures peak vertical load	No	No	Yes	No
Measures nominal lateral load	Yes	Yes	Yes	Yes
Measures peak lateral load	No	No	Yes	No
Measures nominal longitudinal load	No	No	No	Yes
Measures in tangent track	Yes	Yes	Yes	Yes
Measures in curved track	Yes	Yes	No	Yes
Supplier of UIUC's data	AAR, TTX	Progressive Rail	Amtrak, Union Pacific	UIUC

4.3 Shared Use Loading Environment in North America

The railroad operating and loading environment in North America is increasingly made up of shared corridors as expanded and improved passenger rail service is added to the existing freight network. Changes in freight railroad infrastructure, rolling stock, and operating practices involving the accommodation of passenger service have introduced many challenges (Caughron et al. 2012). One of these challenges is the design and performance of critical infrastructure components. Because of the diverse nature of the wheel loads and speeds on shared-use infrastructure, designing components within the track structure requires significant analysis. Most design decisions cannot be made without gaining a quantitative understanding of the entire load spectrum. To better understand the loads applied to the infrastructure, UIUC has acquired WILD data from Amtrak's Northeast Corridor (a shared corridor in operation for many decades) and the Union Pacific Railroad (UPRR) (Figure 4.4). Figure 4.5 illustrates how loads can vary on shared use infrastructure, even within particular vehicle types. Figure 4.6 shows the wide variation of loads on a heavy haul freight line.

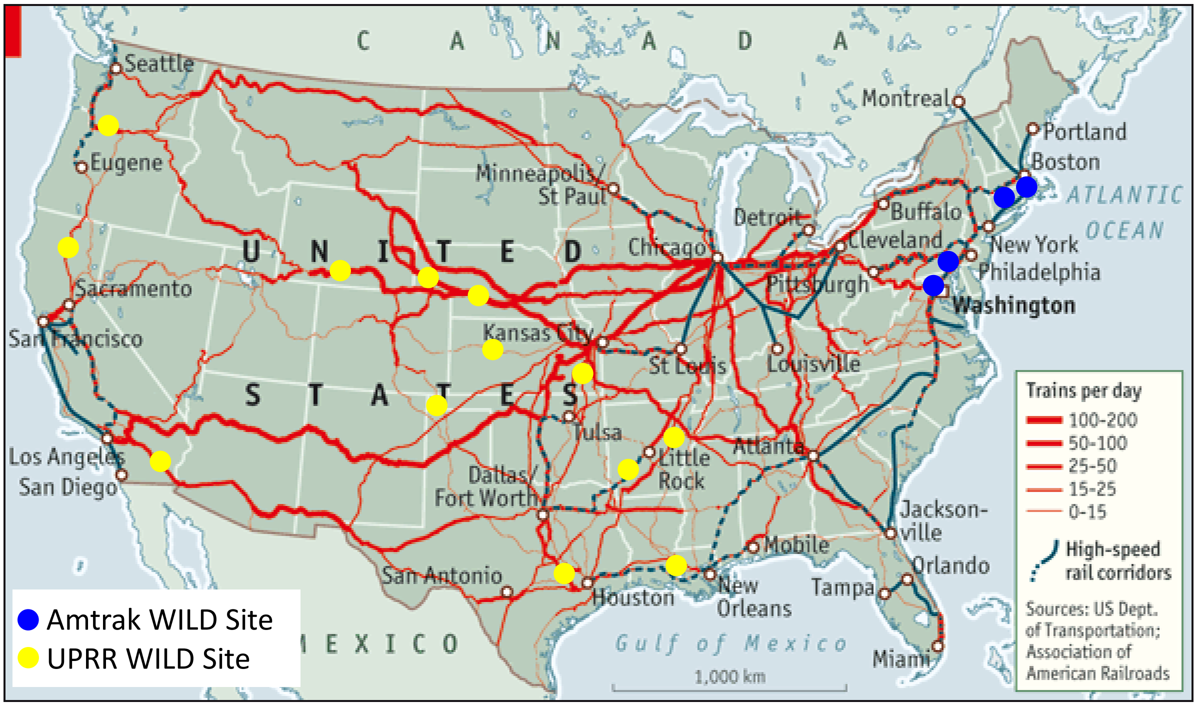

Figure 4.4 WILD data provided to UIUC by Amtrak and UPRR

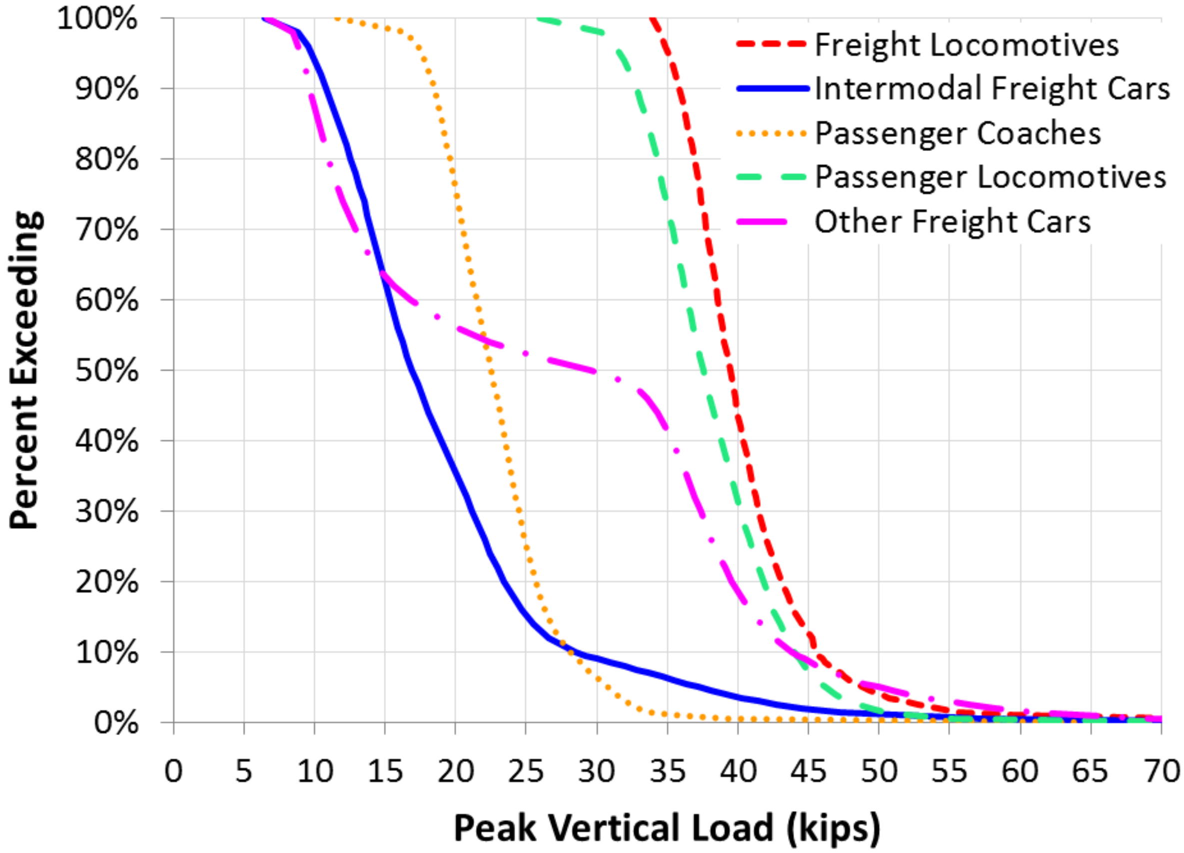

Figure 4.5 Percent exceeding particular peak vertical loads on Amtrak at Edgewood, Maryland (WILD data from November 2010)

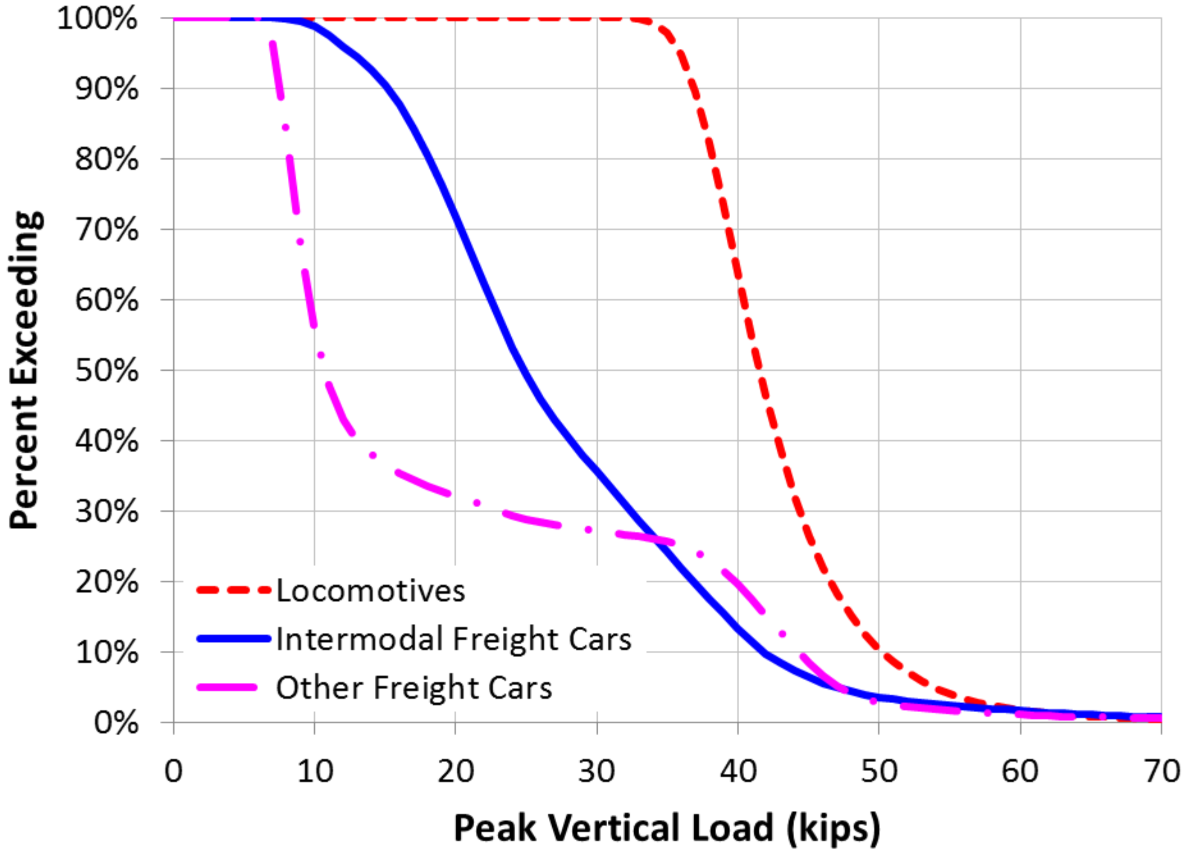

Figure 4.6 Percent exceeding particular peak vertical loads on UPRR at Gothenburg, Nebraska (WILD data from January 2010)

Tables 4.5 and 4.6 provide tabular depictions of the static and peak load spectrums that represent the diverse rolling stock composition in North America. For the purposes of this summary and any following figures that reference them, “unloaded freight cars” are considered to be any non-intermodal freight car whose nominal wheel load is 15 kips or less.

Some statistical testing was performed to determine if one month was representative of the entire population of wheel loading. A series of Kolmogorov-Smirnov tests were performed to compare wheel load data from multiple months. When the entire data set was used (greater than 140,000 wheels per month), there was a statistically significant difference in months because the sample size effectively captured the entire population. When the sample size was reduced to about 2,000 random wheels per month (which still provided an adequate representation of the data), the month-to-month variation was not statistically significant. Therefore, one month's worth of data can be used to make broader generalizations of the wheel load data.

Table 4.5 Distribution of static wheel loads

	Nominal Load (kips)								
Car Type	Mean	10%	50%	75%	90%	95%	97.5%	99.5%	100%
Unloaded Freight Car[4]	6.6	5.2	6.2	7.2	8.5	9.6	11.0	13.6	15.0
Loaded Freight Car[4]	33.4	24.3	34.8	37.1	38.7	39.5	40.2	41.4	45.5
Intermodal Freight Car[4]	20.5	10.4	18.8	26.8	32.9	35.3	36.8	39.8	50.6
Freight Locomotive[4]	33.6	31.4	33.6	34.8	35.9	36.6	37.2	38.5	43.5
Passenger Locomotive[5]	27.0	23.3	26.1	28.4	33.5	35.8	37.2	39.3	42.6
Passenger Coach[5]	15.0	12.7	14.7	16.4	17.7	18.3	19.0	20.1	45.4

Table 4.6 Distribution of peak wheel loads

	Peak Load (kips)								
Car Type	Mean	10%	50%	75%	90%	95%	97.5%	99.5%	100%
Unloaded Freight Car[4]	10.8	7.4	9.2	11.2	15.8	20.5	26.4	39.7	100.8
Loaded Freight Car[4]	42.3	32.6	42.3	45.6	49.8	56.2	65.3	84.7	156.6
Intermodal Freight Car[4]	27.5	15.2	24.8	34.6	41.9	46.8	54.3	74.8	141.9
Freight Locomotive[4]	42.8	36.9	41.6	45.3	50.1	53.9	57.5	68.8	109.6
Passenger Locomotive[5]	38.1	31.1	36.7	41.5	46.4	50.0	53.6	63.4	94.0
Passenger Coach[5]	23.2	17.5	21.7	25.0	30.2	35.3	42.9	58.5	108.8

4.4 Sources of Load Variation

Wheel loads vary due to many causes, including, but not limited to, static load, speed, temperature, location, position within the train, vehicle characteristics, track geometry and quality, curvature, and grade. Because WILDs are constructed on tangent track, and they are dispersed throughout the United States, they are able to capture many of these sources of variation.

[4] Source of data: Union Pacific Railroad; Gothenburg, Nebraska; January 2010
[5] Source of data: Amtrak; Edgewood, Maryland, Hook, Pennsylvania, and Mansfield, Massachusetts; November 2010

4.4.1 Static Wheel Load

The nominal (static) wheel load is the best indicator of the load expected to enter into the track structure and is highly dependent on the type of vehicle passing over the WILD. Vehicles with higher nominal wheel loads produce higher peak wheel loads, as shown in Figure 4.7. Density contours are displayed to show areas of high data concentration. The wide distribution beyond the most highly concentrated data, however, suggests that there are other factors affecting the peak load entering the track structure.

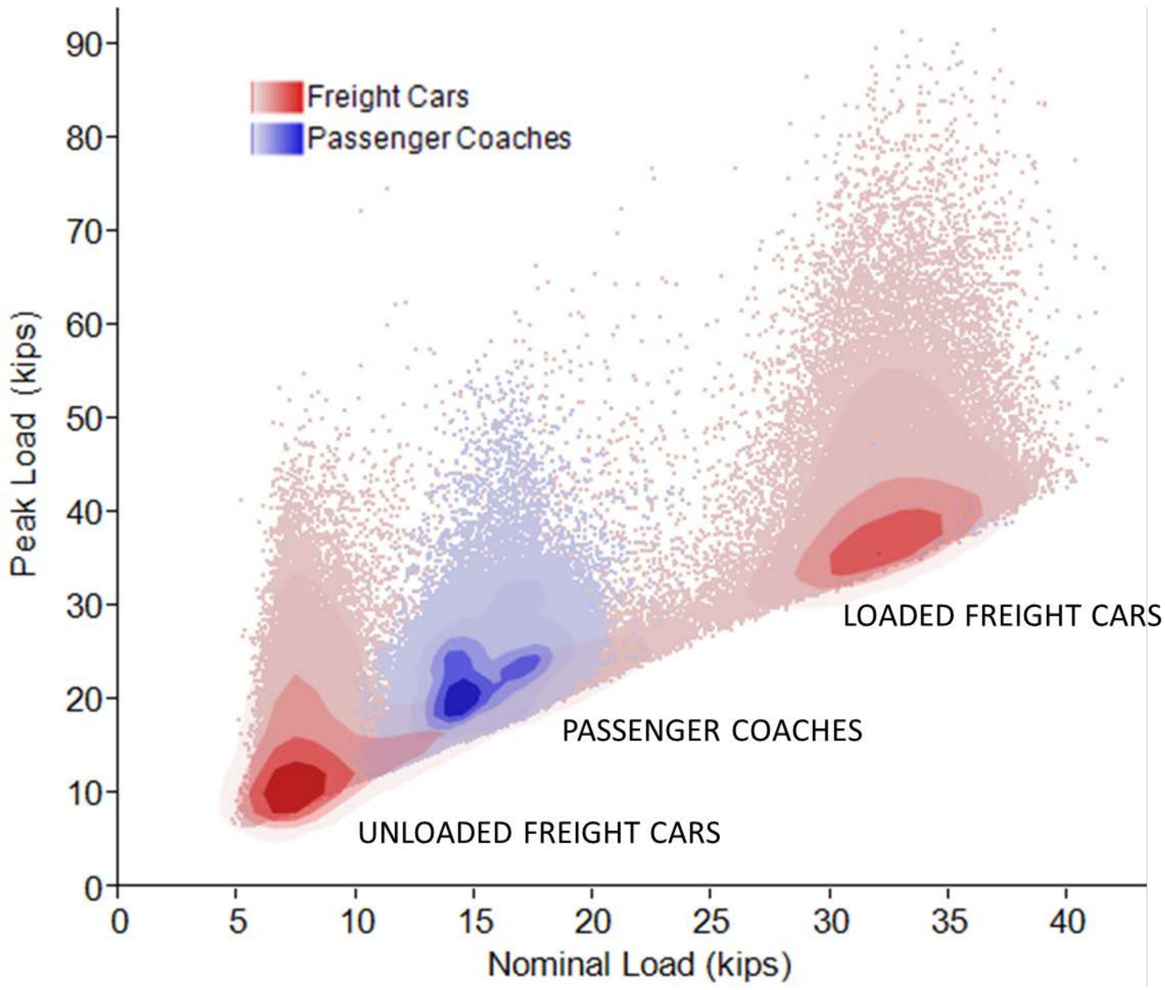

Figure 4.7 Effect of car type on peak load on Amtrak at Edgewood, Maryland (WILD data from November 2010)

4.4.2 Speed

Field observations suggest that loads at the wheel-rail interface produced by moving loads are greater than those produced by the same wheel loads at rest (Kerr 2003). Specifically, dynamic loads can be produced by roll, slip, lurch, shock, buff, torque, load transfer, vibration, and unequal distribution of lading within the rolling stock (Hay 1982). Generally, dynamic and impact forces can be caused by imperfections in the moving vehicles (as listed above), track geometry irregularities, and variations in track stiffness (Kerr 2003). However, the relationship between speed and total vertical load is not easily quantified or characterized. As shown in Figure 4.8, the majority of the peak vertical wheel loads exhibit minimal increases with increased speed. Figure 4.9 shows a similar relationship with much higher maximum speeds. This increase may simply be due to dynamic interaction between the naturally-oscillating vehicles and the track (Esveld 2001). The effect of speed on total vertical load is further explored in Chapter 5.

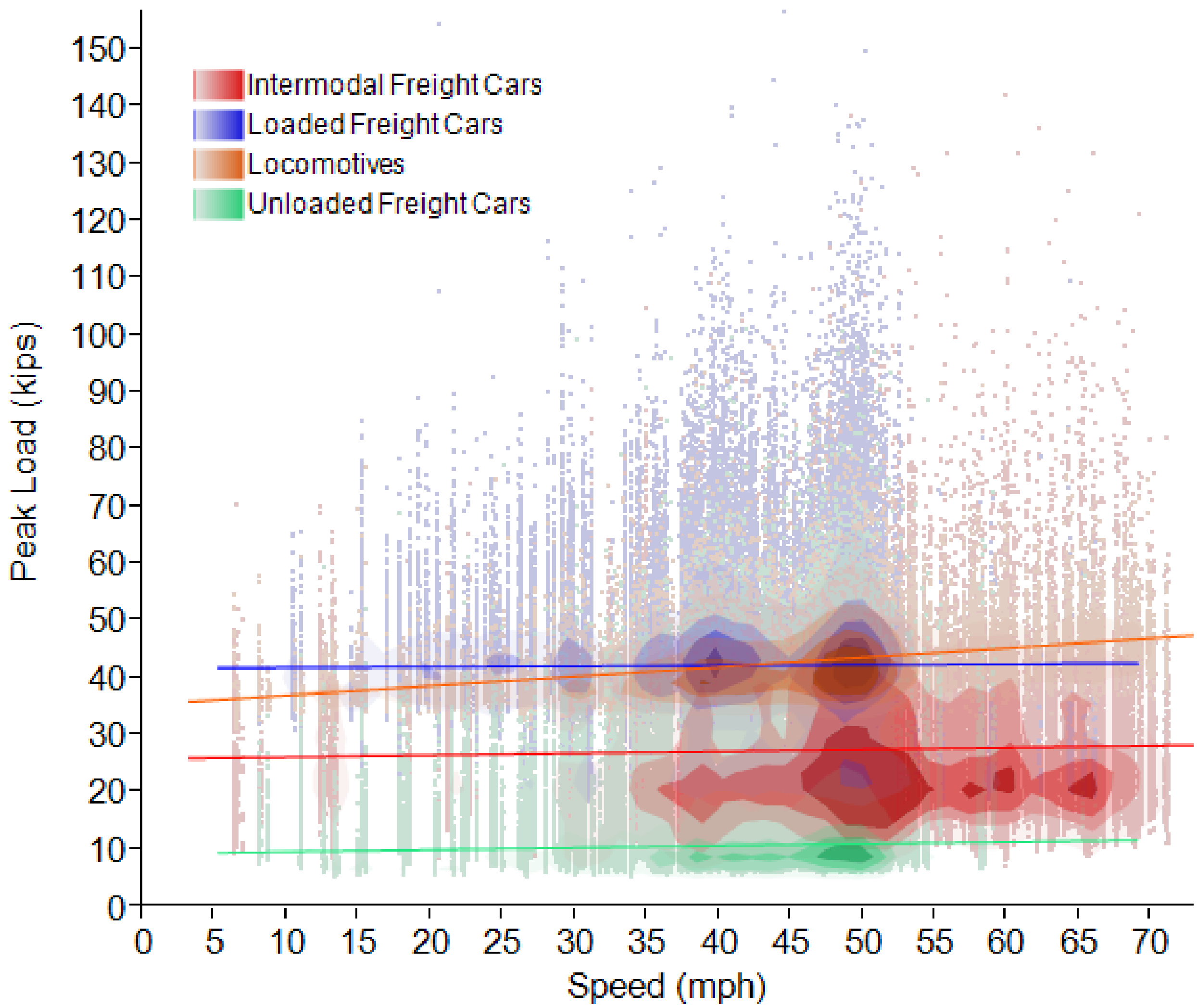

Figure 4.8 Effect of speed on peak load on UPRR at Gothenburg, Nebraska (WILD data from January 2010)

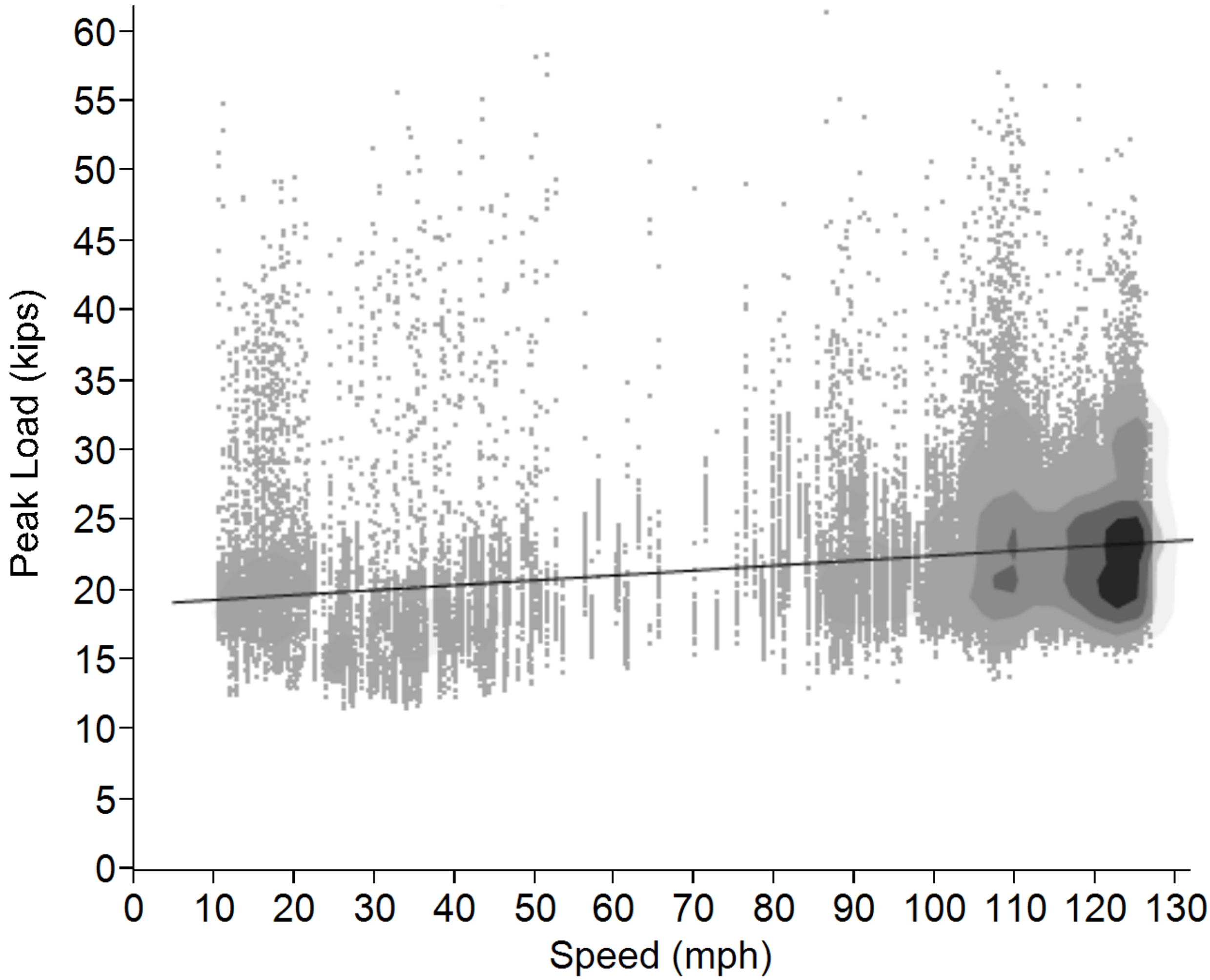

Figure 4.9 Effect of speed on peak load on Amtrak at Edgewood, Maryland (Passenger WILD data from November 2010)

4.4.3 WILD Site Location

The location of the WILD site provides another very significant source of variation in loads. Each site sees different distributions of car types and operating speeds. These varied traffic characteristics often produce widely varied loads at the wheel-rail interface. To illustrate this, Figure 4.10 compares non-intermodal freight traffic at Martin Bay, NE (where 99% of all wheels exceed 30 kips) with that at Elton, LA (where only 48% of all wheels exceed 30 kips). Figure 4.10 also illustrates the different load magnitudes associated with loaded and unloaded freight cars, indicated by the steepest portions of the Elton curve. It appears as if only loaded freight cars pass the Martin Bay WILD, causing significant deviation from a distribution that includes unloaded cars as well.

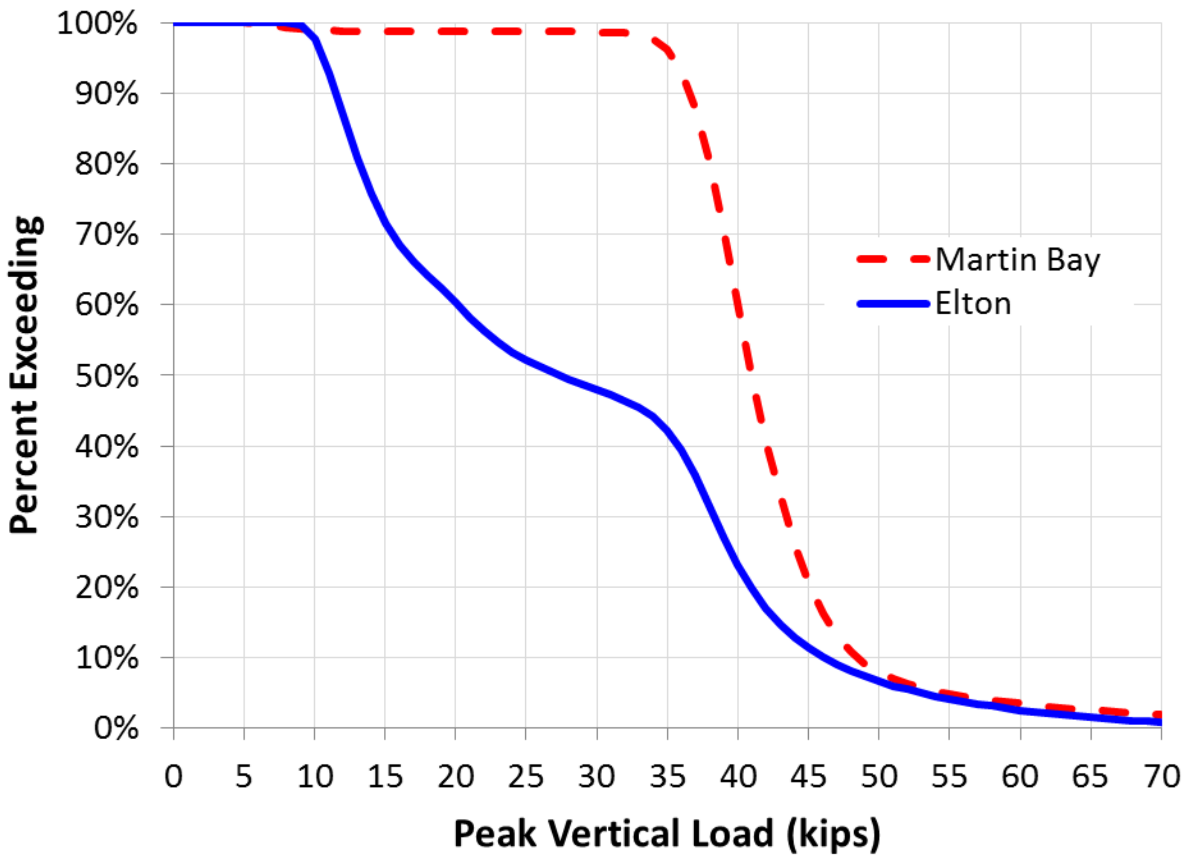

Figure 4.10 Variation of peak vertical loads between Martin Bay, Nebraska and Elton, Louisiana (non-intermodal freight car WILD data from January 2010)

The variation depicted in Figure 4.10 is to be expected, as these two WILD sites are in different regions of the country and have vastly different traffic compositions. However, WILD sites in the same region on infrastructure owned by one railroad can also exhibit substantial differences in loading. Figure 4.11 illustrates passenger coach wheel loads from four sites along Amtrak's Northeast Corridor. While each distribution represents passenger coaches, there are multiple types of passenger coaches at each site, adding further variation within traffic type. Each site experiences commuter service (with different types of equipment) and Amtrak regional service, while Mansfield (150 mph), Edgewood (135 mph), and Hook (110 mph) experience higher-speed Acela Express service. Each of these operating services uses different types of equipment, resulting in substantial variability even within a particular traffic type (i.e.

passenger coaches). As shown in the figure, just 5% of the peak wheel loads captured at Hook exceed 25 kips, while almost 57% of the wheels passing over the Mansfield site produce peak loads in excess of 25 kips. The compositions of passenger traffic at these two sites are similar, yet there are evidently other sources of variability affecting the distribution of peak wheel loads.

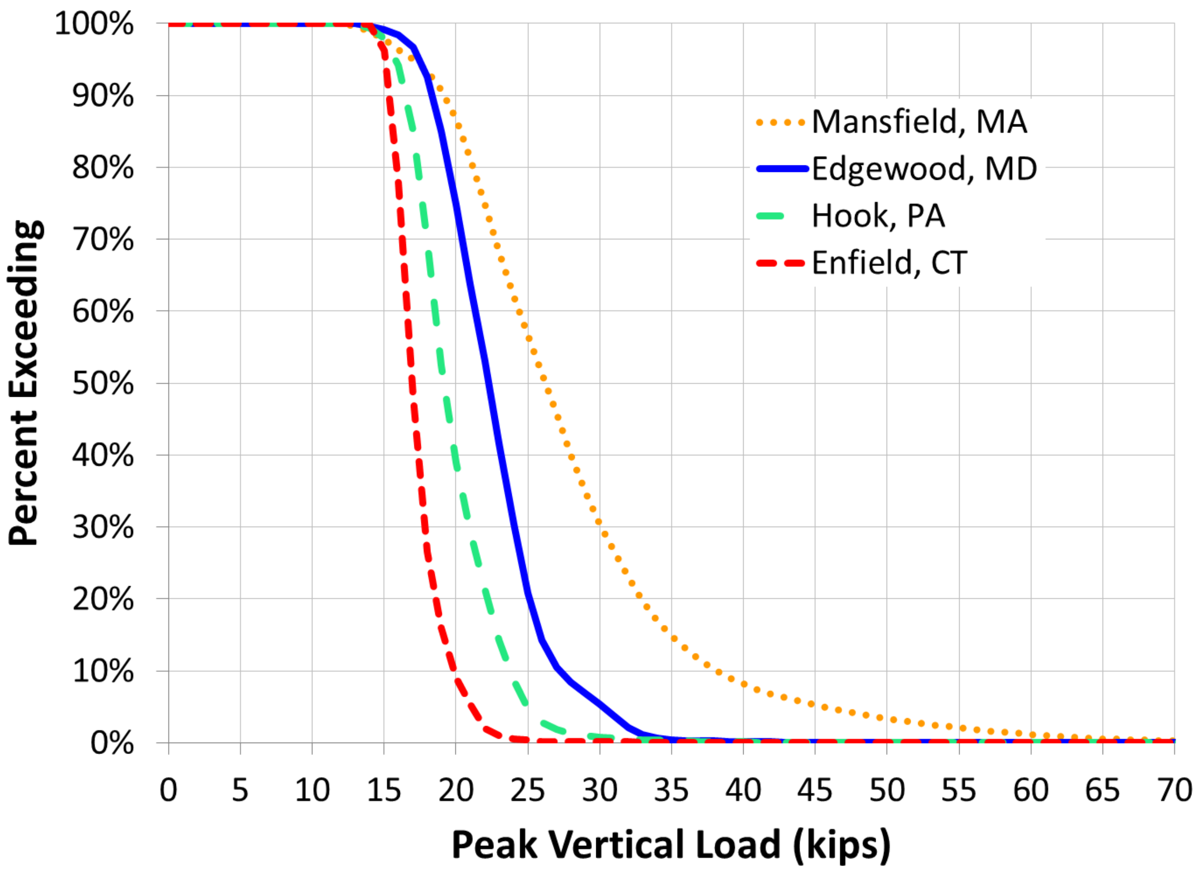

Figure 4.11 Variation of peak vertical loads along Amtrak's Northeast Corridor (passenger car WILD data from April 2011)

4.4.4 Month within the Year

While it has already been shown that there is variability across sites due to varying traffic characteristics, there also exists seasonal variability in loading at a single site. According to Kerr, when the track substructure is frozen, it becomes stiffer and causes higher loads at the wheel-rail interface (Kerr 2003). The condition of the wheel may also deteriorate during the winter months due to a harsher braking environment. In fact, certain conditions, including frozen ballast and subgrade, can result in up to a ninefold increase in track stiffness from freshly-tamped track (Kerr 2003). Cold weather can also stiffen various damping components within the carbody (Nurmikolu et al. 2013) and perhaps the track superstructure, further increasing the wheel load. One would then expect significant variability in loads according to seasonal changes. In fact, UPRR has collected WILD data showing a clear increase in the number of severe impacts during the winter months on its network (GeMeiner 2005).

Generally, month-to-month variability at a particular site is actually quite minimal. A brief review of the static wheel loads collected during multiple months indicates that the rolling stock traveling over the WILD sites remains relatively constant regardless of the month. Compared to other sites and other years within the data provided by UPRR, Figure 4.12 depicts relatively large month-to-month variability in peak loads experienced at the Gothenburg, Nebraska WILD site. However, the loads do not follow the expected trend (higher wheel loads during the colder months) according to monthly temperature fluctuations at a location that sees significant seasonal temperature variation. Therefore, there doesn’t appear to be enough evidence to conclude that seasonal variations affect the general shape of the wheel load distribution.

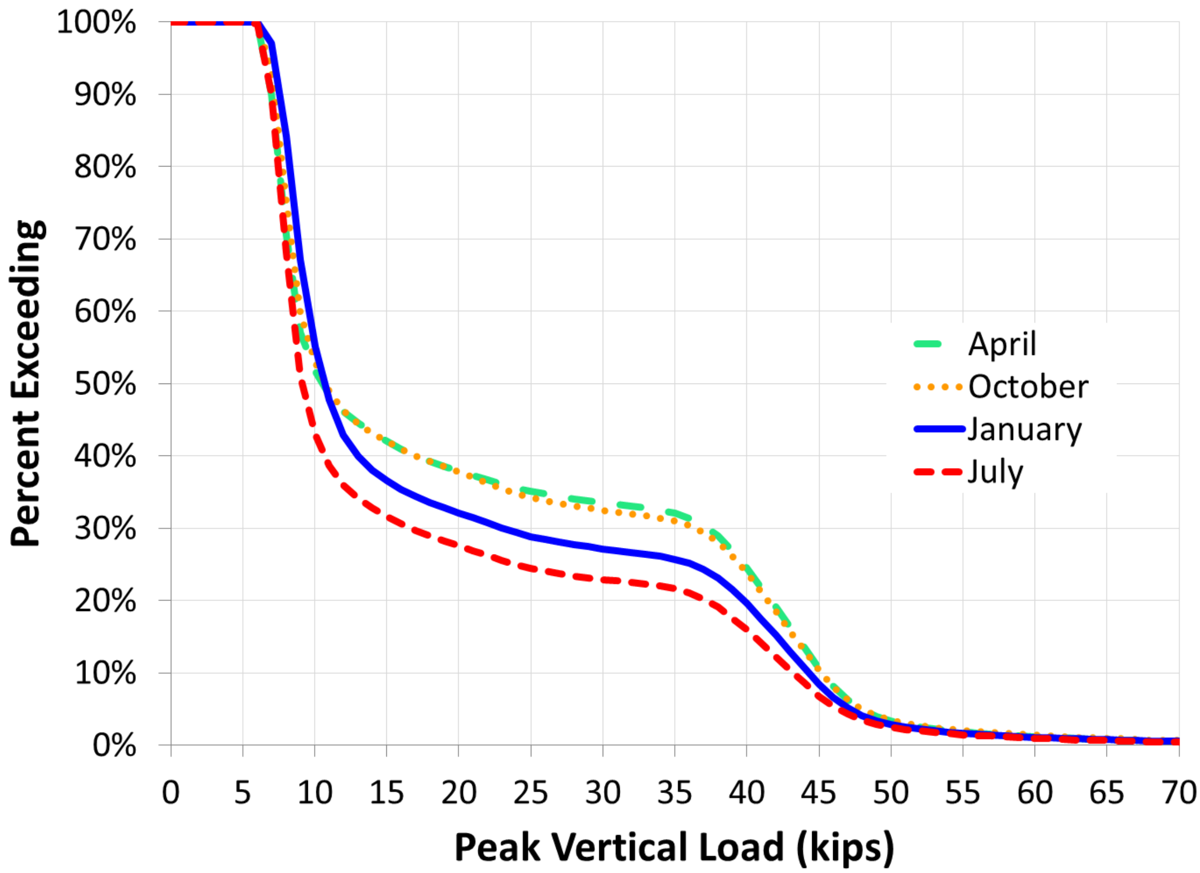

Figure 4.12 Monthly variation of peak vertical loads on UPRR at Gothenburg, Nebraska (non-intermodal freight car WILD data from 2010)

However, focusing on the highest loads provides some clarity regarding the most severe impacts, as shown in Figure 4.13. The highest 0.1% of peak vertical loads in January is higher than the most severe impact loads recorded during the warmer months. This observation is consistent across both operators (Amtrak and UPRR) and multiple WILD sites (locations where substantial seasonal temperature fluctuations would occur), confirming the hypothesis that the stiffer track structure (higher track modulus) resulting from colder temperatures does not attenuate the high impact loads as well as a more flexible track structure (lower track modulus).

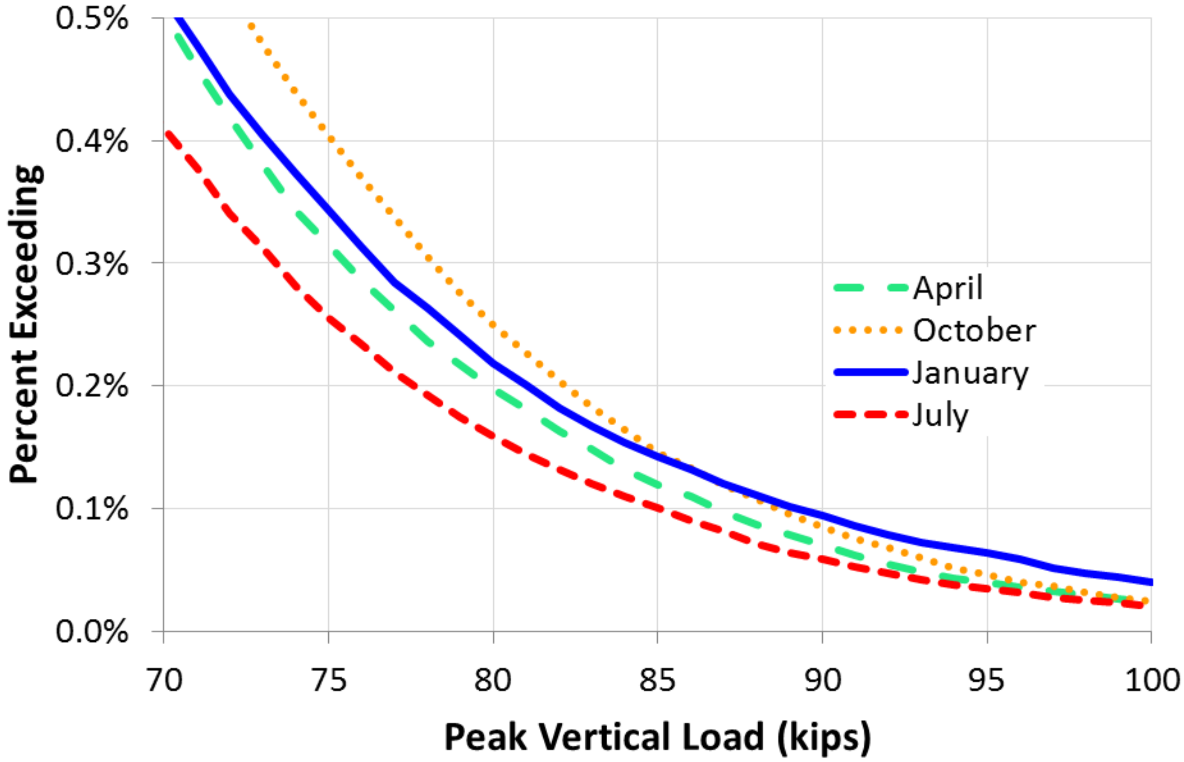

Figure 4.13 Monthly variation of highest peak vertical loads on UPRR at Gothenburg, Nebraska (non-intermodal freight car WILD data from 2010)

4.4.5 Wheel Irregularities

Perhaps the greatest contributor to increases in loads entering the track structure as detected by the WILD is the condition of the wheel. Irregularities on the wheel can result in impacts that severely damage the rail and other components of the track structure. For instance, a 100-kip impact resulting from a flat wheel can increase the contact stress in the rail by up to 200% (GeMeiner 2005). Therefore, variability in the quality of wheels traveling over the infrastructure creates substantial variation in the loads entering that structure. Figure 4.14 shows peak wheel load as a function of speed for passenger coach data on Amtrak's Northeast Corridor. The substantial number of wheel loads exceeding 50 kips at roughly half the maximum speed suggests a high volume of poorly-performing wheels travelling over this WILD site. These wheels are imparting loads up to six times their static load into the track structure, increasing the potential for damage to the rail and other track components. The condition of these wheels may contribute to the site-specific diversity as shown in Figure 4.11.

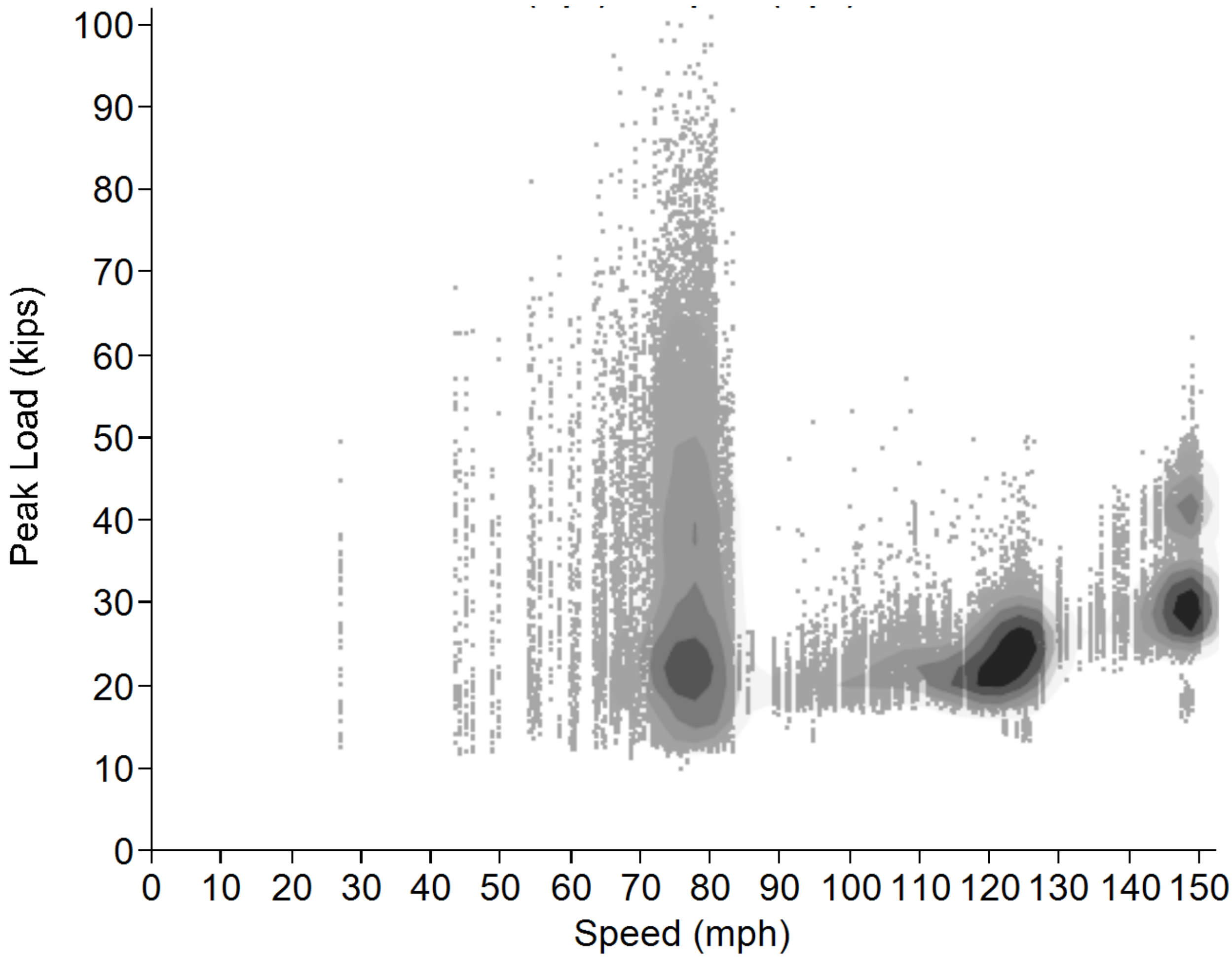

Figure 4.14 Effect of wheel condition on peak vertical load on Amtrak at Mansfield, Massachusetts (passenger WILD data from November 2010)

4.4.6 Other Sources of Variability

Because the WILD is installed on high-quality tangent track, the effect of wheel position within the truck, car, or train may not be fully realized. It is well understood, though, that the leading axle of any particular truck will create the highest lateral loads within a curve (Andersson et al. 2013). In distributed power applications with curvature and gradients, there is also substantial variation along the length of the train in lateral and longitudinal wheel loads (Peltz 2013). In the future, the UIUC research team will further test this hypothesis using both WILD and IWS data to determine what effect, if any, the axle's position within the rolling stock has on the loading environment.

The effect of curvature and grade are also not clear from WILD data due to the detector's characteristics. Curvature substantially affects the lateral loads applied by the wheel and, along with gradients, can also cause variation in vertical loads (Figure 4.15).

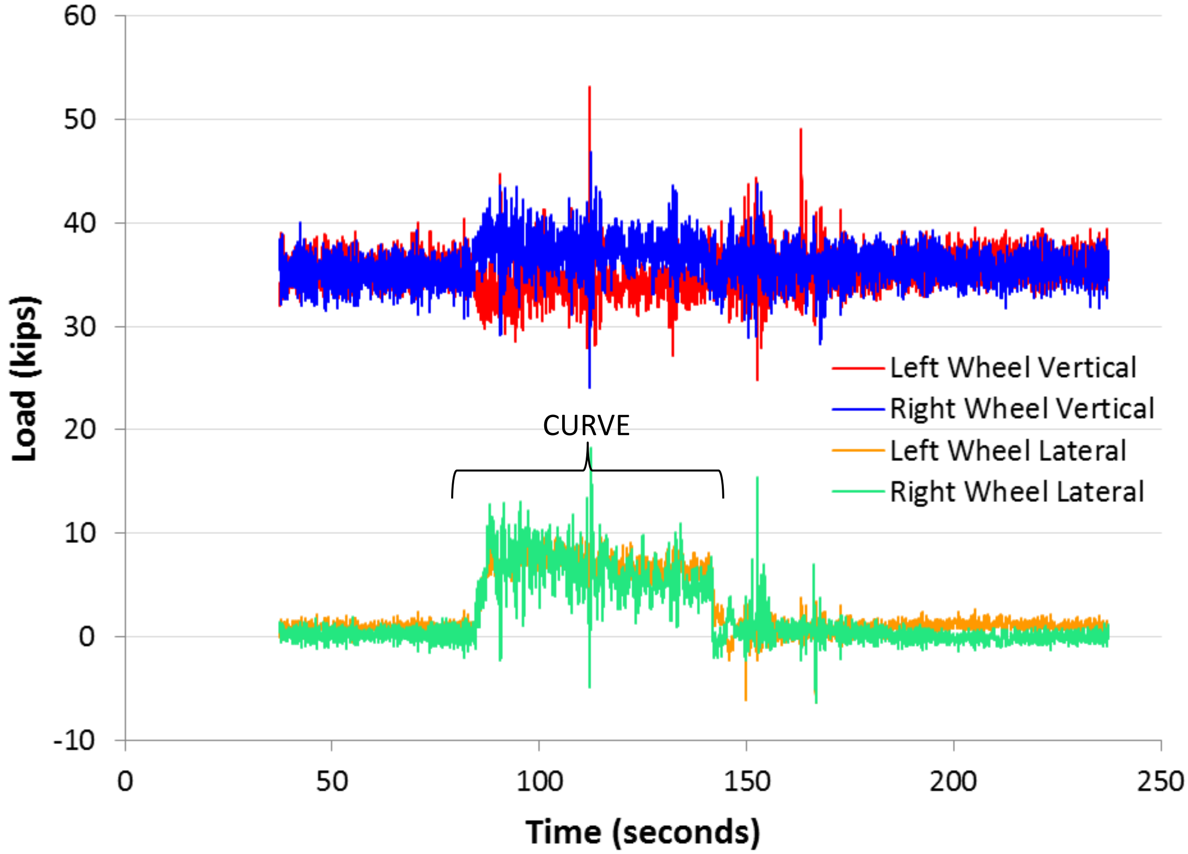

Figure 4.15 Vertical and lateral wheel loads in a left-handed curve on UPRR (IWS data from March 2006)

As shown in Figure 4.15, the vertical load created by the outside wheel increases during the curve, while the vertical load from the inside wheel decreases in the curve section. Furthermore, the lateral loads from both wheels increase significantly in the curved portion of the track when compared to the tangent sections. However, the lateral load decreases throughout the duration of the curve because the train is slowing down as it travels through the curve. To better understand the effect of speed on the lateral wheel loads in a curve, the degree of curvature and superelevation must be considered. Cant deficiency, which is the difference between equilibrium superelevation and actual superelevation in a curve (Andersson et al. 2013), considers degree of curvature, curve superelevation, and vehicle speed and can be expressed as follows:

$$h_d = \frac{2b_0}{g}\left(\frac{v^2}{1746.40/D}\right) - h_t$$

where, h_d = cant deficiency (mm)

$2b_0$ = distance between contact patches on a wheel set (assumed 1,500 mm)

g = acceleration due to gravity (9.81 m/s^2)

v = vehicle speed (m/s)

D = degree of curvature

h_t = actual superelevation of curve (mm)

Relating lateral wheel load magnitudes to cant deficiency allows different curves with different balance speeds to be more effectively compared. Figure 4.16 shows the relationship between cant deficiency and lateral wheel loads on the same left-handed curve illustrated in Figure 4.15.

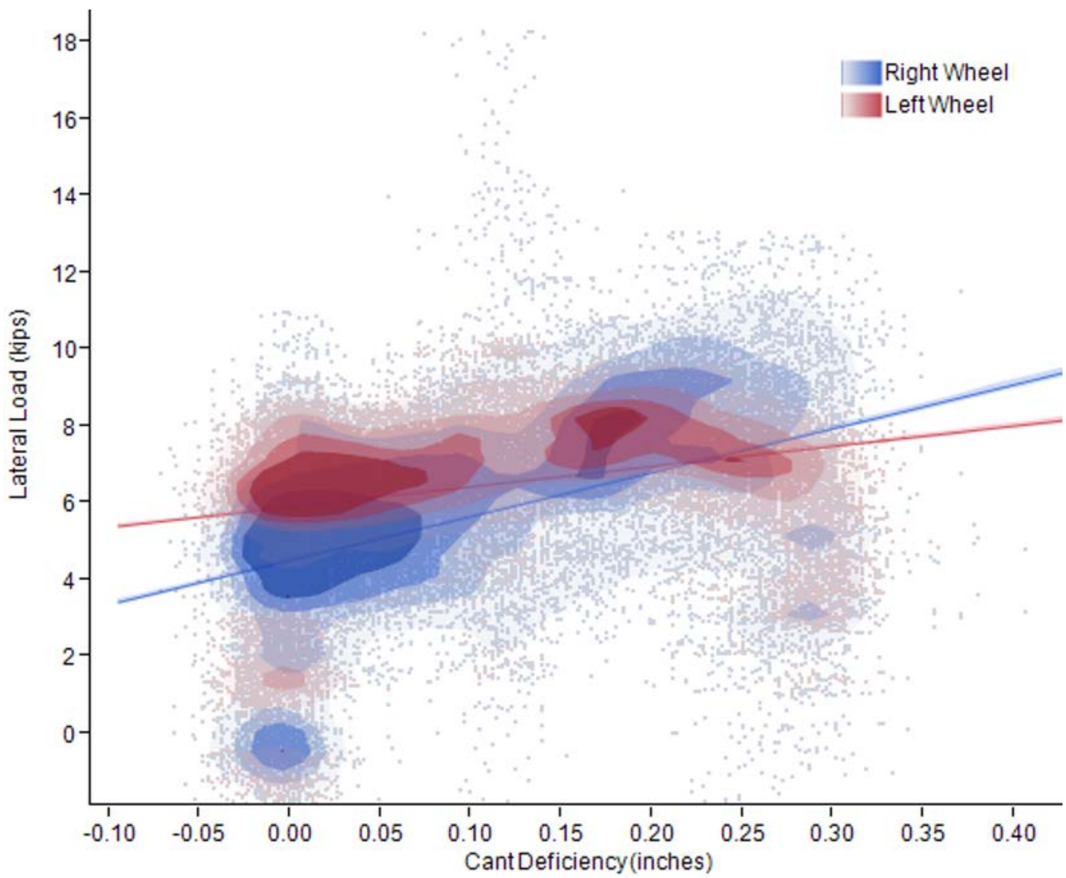

Figure 4.16 Effect of cant deficiency on lateral loads in curved track on UPRR (IWS data from March 2006)

Because the instrumented wheel set is installed on a standard, relatively stiff truck, the lateral forces from both wheels increase with increased cant deficiency (a function of increased speed). The rate at which the right (outer) wheel increases is higher partially due to increased centrifugal forces at higher speeds, but mostly due to higher angle of attack (yaw angle). In the future, UIUC will utilize TPD data to explore the relationship between angle of attack and the magnitude of lateral loads entering the rail in curved track.

4.5 Conclusions

The data collected at the Amtrak and UPRR WILD sites provide unique insight into the loading trends of the rolling stock travelling over each of these networks. Specifically, these data provide insight on primarily passenger operations, primarily freight operations, and true shared-use operations. Therefore the following conclusions can be roughly applied for each of these situations across North America:

- The WILD is a useful tool for collecting and analyzing data about loads entering the track structure
- Vehicle type and its associated static load provides a baseline for the expected total load at the wheel-rail interface
- Increasing speed minimally increases the most common wheel loads; however, severe impact loads become much more severe at higher speeds
- Traffic composition and other site-specific parameters play a substantial role in the distribution of the loading environment
- Seasonal effects in load variation, while greatly contributing to the magnitude of severe impacts, minimally affect the majority of the wheel load distribution
- Wheel condition is a substantial factor in determining peak loads entering the track structure
- Lateral loads on both rails increase with increased cant deficiency on curved track

Identifying the sources of wheel load variation, as well as determining relationships between parameters that incorporate multiple data collection methods, will more accurately capture the loading environment. This will lead to improvements in design and performance of critical infrastructure components and the entire track structure.

The effects of speed and wheel condition are especially important in quantifying loads at the wheel-rail interface. There have been numerous attempts to quantify these effects, typically using a dynamic or impact factor that is applied to an expected static load. However, many of these factors were developed using older operating conditions or at locations with traffic that is not representative of the North American network. Therefore, these particular factors will be further evaluated and quantified in Chapter 5.

CHAPTER 5: EVALUATION OF DYNAMIC AND IMPACT WHEEL LOAD FACTORS AND THEIR APPLICATION FOR DESIGN

5.1 Introduction

As discussed in Chapter 4, there are many parameters that contribute to the actual load imparted into the track structure from the car body. Some of these parameters are considered in design by using a dynamic factor or impact factor for more accurate load estimation. Both of these factors will be defined and evaluated using actual wheel loading data in this chapter.

There are several types of loads that can be used to design the track structure: static, quasi-static, dynamic, and impact loads. The static load is simply the weight of the rail vehicle at rest. The quasi-static load can be considered the combined static load and the effect of the static load at speed, independent of time (Standards Australia International 2003). The quasi-static load is perhaps best illustrated in curved track, where the vehicle imparts loads onto the rail due to centripetal force and curving (Andersson et al. 2013). The dynamic load is the additional load (above static load) due to high-frequency effects of wheel/rail load interaction, considering track component response and involving inertia, damping, stiffness, and mass. This load is more difficult to quantify because it is characterized by highly variable load inputs dependent on time. The impact load, which often creates the highest loads in the track structure, is created by track and vehicle irregularities. These impacts create high-frequency, short-duration loads that travel through the infrastructure and can lead to substantial damage.

5.2 Identification and Evaluation of Dynamic Wheel Load Factors

It is well understood that forces at the wheel-rail interface produced by moving loads are greater than those produced by the same wheel loads at rest (Kerr 2003). Typically, therefore, the design wheel load is higher than the static wheel load to account for this increase due to speed, i.e.,

$$P_d = \phi P_s$$

where, P_d = dynamic wheel load

ϕ = dynamic wheel load factor

P_s = static wheel load

The dynamic wheel load factor is typically developed empirically using field data and is expressed in terms of train speed. The number of elements considered in its development can depend on the sophistication of the track instrumentation implemented and the assumptions made (Doyle 1980). Historically, there have been many efforts undertaken to quantify the increase of load expected at the wheel-rail interface due to speed.

5.2.1 Previous Dynamic Factors

Doyle (1980) provides a summary of many dynamic wheel load factors. Several factors are calculated using only train speed. Beginning in 1943, the Deutsche Bahn (Germany Railways) began using an equation that is only valid for speeds up to 200 kph (125 mph) (Schramm 1961). In 1968, a dynamic factor was prepared for the Washington Metropolitan Area Transit Authority (WMATA) and used in subsequent recommended standards for transit trackwork (Prause et al. 1974). More recently, another speed-dependent dynamic factor was developed in Iran (Sadeghi & Barati 2010). The final factor dependent only on train speed, although not applied at the wheel-rail interface, is included because of its importance in the design of the track structure. The Speed Factor found in Chapter 30 of the AREMA Manual (AREMA C30) is used as part of the flexural design of concrete crossties after a distribution factor and impact factor (described in Section 5.3) are applied to a single wheel load (American Railway Engineering and Maintenance-of-Way Association 2012). The Chapter 30 Speed Factor, developed in the early 1980s by the AREMA Committee, is constant below 20 mph and above 120 mph (McQueen 2010).

Most of the dynamic factors, however, have been developed to incorporate additional parameters beyond train speed. A. N. Talbot provided a factor to the American Railway Engineering Association (AREA) based on tests his committee conducted in the 1910s (Hay 1953). The Talbot dynamic factor incorporates wheel diameter and is still used in modern North American track analysis (American Railway Engineering and Maintenance-of-Way Association 2012). The South African Railways formula is similar to the Talbot formula, but is calculated for narrow gauge track. The Indian Railways dynamic factor incorporates track modulus as an indicator of track condition (Srinivasan 1969), while the Clarke Formula algebraically combines the Talbot and Indian Railways dynamic factors (Doyle 1980).

Three additional dynamic factors have been developed that incorporate many other parameters. The Eisenmann dynamic factor incorporates the condition of the track and uses a statistical approach where the rail bending stresses and deflections are normally distributed and calculated using Zimmermann's longitudinal beam model (Esveld 2001). The British Railways dynamic factor is used for discrete irregularities, such as a dipped rail joint, and was developed in the 1970s using specific track infrastructure, incorporating the vehicle's unsprung mass, track stiffness at the irregularity, and speed. The most comprehensive dynamic factor was developed by the Office of Research and Experiments (ORE) of the International Union of Railways (UIC), particularly Birmann. This factor incorporates the track geometry, vehicle suspension, vehicle speed, vehicle center of gravity, age of track, curve radius, superelevation, and cant deficiency. Due to the lack of experimental data related to each of these parameters, Doyle (1980) makes some reasonable assumptions and simplifies parts of the factor accordingly.

A comparison of vehicle and track parameters included in each of the dynamic factors is shown in Tables 5.1 and 5.2, while Figures 5.1a and 5.1b display the design dynamic factors increasing with speed. Previous research has shown that the rate of load increase due to speed is much higher when wheel quality is poor (Van Dyk et al. 2013b).

Table 5.1 Summary of Dynamic Factors (adapted from Doyle (1980))

		Vehicle Parameters Included						Track Parameters Included					
Dynamic Factor	Expression for ϕ	Train Speed	Wheel Diameter	Static Wheel Load	Unsprung Mass	Vehicle Center of Gravity	Locomotive Maintenance Condition	Track Modulus	Track Stiffness at Rail Joint	Track Joint Dip Angle	Cant Deficiency in Curves	Curve Radius	Track Maintenance Condition
Talbot (Hay 1953)	$1+\frac{33V}{100D}$	●	●										
Indian Railways (Srinivasan 1969)	$1+\frac{V}{3\sqrt{U}}$	●						●					
Eisenmann (Esveld 2001)	$1+\delta\eta t$	●											●
ORE/Birmann (Birmann 1965)	$1+\alpha+\beta+\gamma$	●				●	●				●	●	●
German Railways (Schramm 1961)	$1+\frac{11.655V^2}{10^5}-\frac{6.252V^3}{10^7}$	●											
British Railways (Doyle 1980)	$1+14.136(\alpha_1+\alpha_2)V\sqrt{\frac{D_jP_u}{g}}$	●		●	●				●	●			
South African Railways (Doyle 1980)	$1+0.312\frac{V}{D}$	●	●										
Clarke (Doyle 1980)	$1+\frac{15V}{D\sqrt{U}}$	●	●					●					
WMATA (Prause et al. 1974)	$(1+0.0001V^2)^{\frac{2}{3}}$	●											
Sadeghi (Sadeghi & Barati 2010)	$1.098+0.00129V+2.59(10^{-6})V^2$	●											
AREMA C30	For $20<V<120: 0.6+0.005V$	●											

Table 5.2 Variable Definitions for Table 5.1

Variable	Definition
V	Train speed (mph)
D	Wheel diameter (in)
U	Track modulus (psi)
δ	0.1, 0.2, 0.3, depending on track conditions
η	1 for vehicle speeds up to 37 mph $1+\frac{V-37}{87}$ for vehicle speeds between 37 and 125 mph
t	0, 1, 2, 3, depending on chosen upper confidence limits defining probability of exceedance
α	Coefficient dependent on level of track, vehicle suspension, and vehicle speed, estimated to be $0.167\left(\frac{V}{100}\right)^3$ in most unfavorable case
β	Coefficient dependent on wheel load shift in curves (0 in tangent track)
γ	Coefficient dependent on vehicle speed, track age, possibility of hanging crossties, vehicle design, and locomotive maintenance conditions, estimated to be $0.10+0.071\left(\frac{V}{100}\right)^3$ in most unfavorable case
$\alpha_1+\alpha_2$	Total rail joint dip angle (radians)
D_j	Track stiffness at the joints (kN/mm)
P_u	Unsprung weight at one wheel (kN)
g	Acceleration due to gravity (m/s^2)

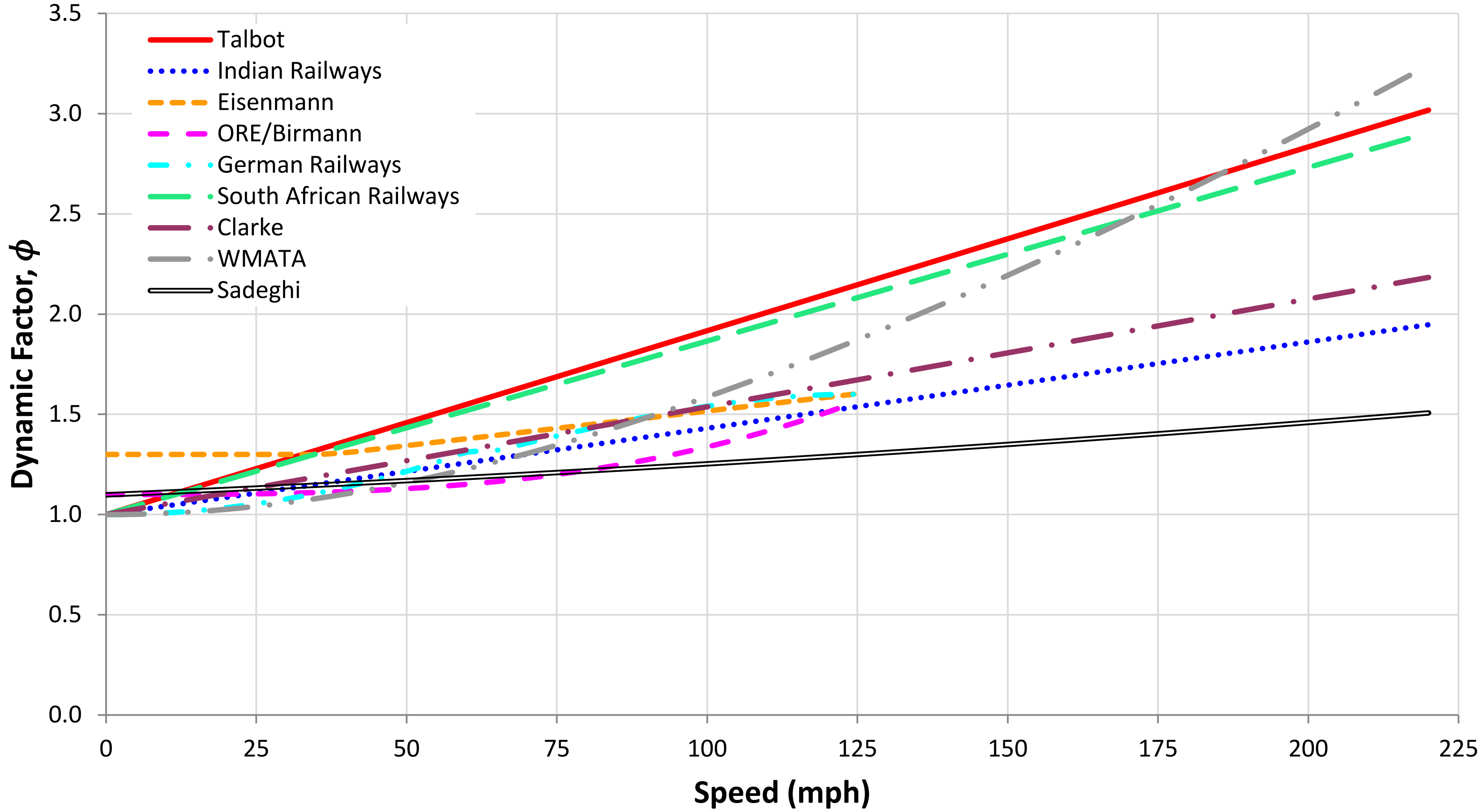

Figure 5.1a Design dynamic factors increasing due to speed

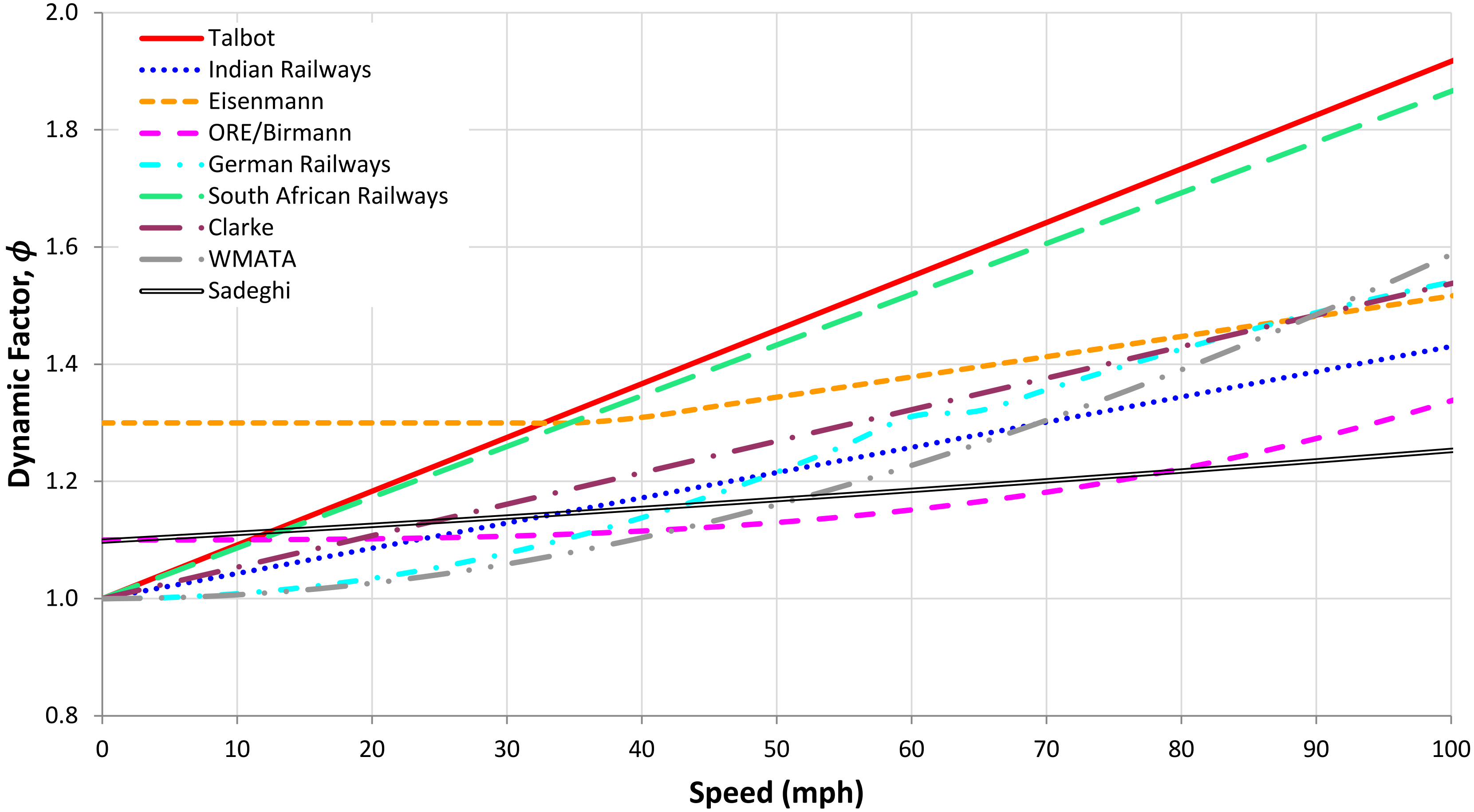

Figure 5.1b Design dynamic factors increasing due to speed

5.2.2 Evaluation of Dynamic Factors

Many of the dynamic factors discussed in the previous section can only be used to predict the load amplification due to speed in specific operating applications. Because they have been developed over many years in different regions of the world, they may not accurately reflect the operating conditions found in North America. To determine the applicability of these formulas to the North American operating environment, wheel impact load detector (WILD) data was used to compare actual loading data to predicted speed-induced gains. Figure 5.2 shows an example of wheel load data to be compared with the plotted dynamic factors. To adequately assess the effectiveness of each of the previously-developed dynamic factors, several evaluative metrics are considered (Table 5.3). The speed-weighted signed difference and load-weighted signed difference were developed to provide a different perspective by weighting train speed and static load respectively.

As discussed in Section 4.2.3, WILD data may underestimate the actual loading conditions because the sites are built with premium components to remove the variation in load due to track geometry and support condition irregularities. However, these data still provide loading information representative of the rail network as a whole and are sufficient for the comparison of dynamic factor effectiveness (Van Dyk et al. 2013a).

It should be noted that two factors have been omitted from this analysis. Because the dynamic factor developed for British Railways is appropriate only at rail joint dips, it is not appropriate to evaluate its effectiveness using WILD data. Because the AREMA speed factor is used in combination with an impact factor and is to be applied as an upper bound at the rail seat, it is not necessarily appropriate to be comparing it with other factors that should be used to predict wheel loads.

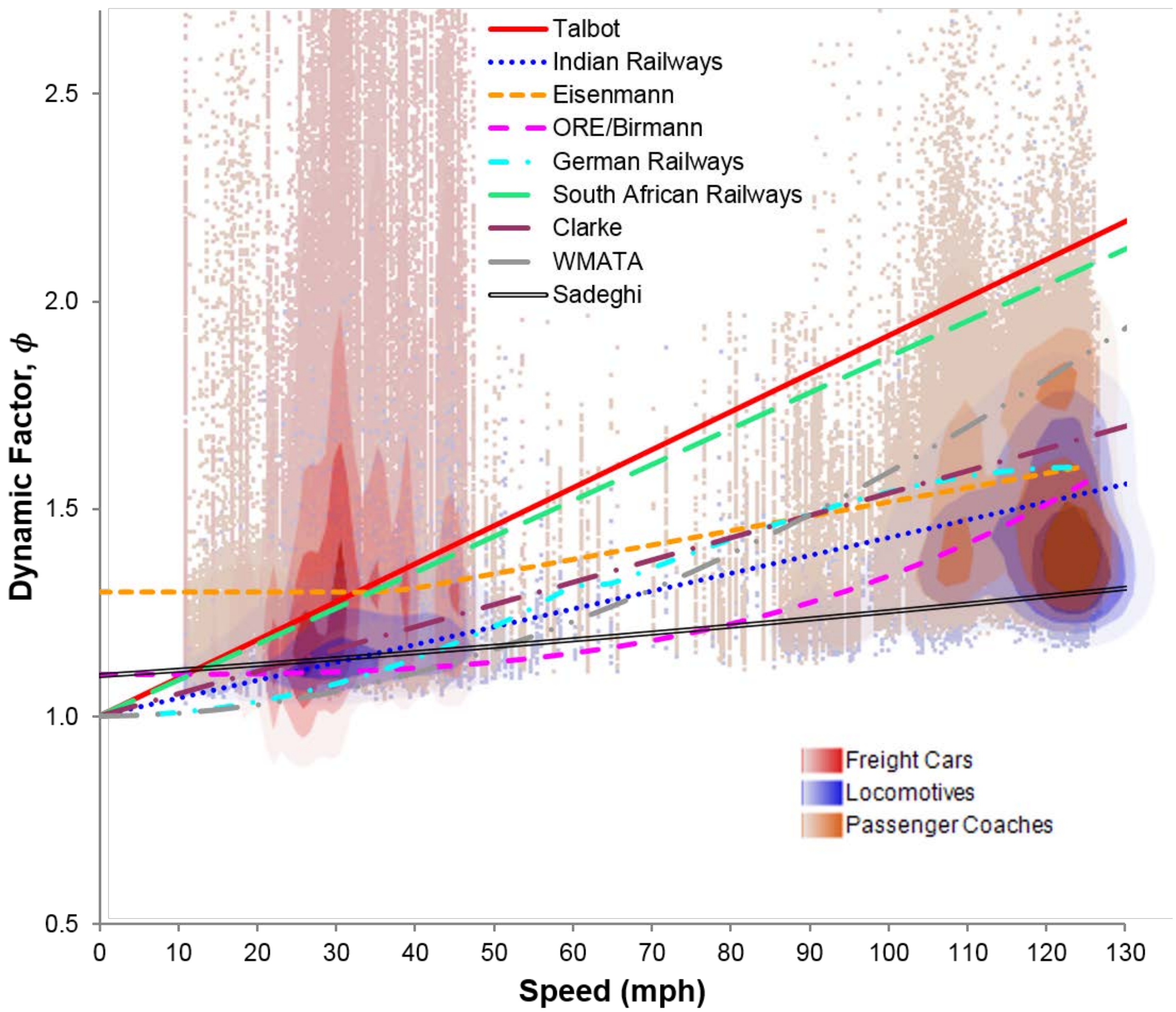

Figure 5.2 Peak/nominal wheel load ratios on Amtrak at Edgewood, Maryland (WILD data from November 2010) and design dynamic factors

Table 5.3 Definitions of dynamic factor evaluative metrics

Percent exceeding – percentage of wheels exceeding predicted dynamic factor

Mean percentage error – computed average of percentage errors by which predictions of a model differ from actual values of the quantity being predicted

$$\frac{100\%}{n}\sum_{i=1}^{n}\frac{f(x_i)-y_i}{y_i}$$

x_i is the speed of a single wheel
y_i is the ratio of peak vertical load to nominal vertical load of a single wheel
$f(x_i)$ is the predicted dynamic factor of a wheel given its speed
n is the total number of wheels

Root mean square deviation – measures differences between values predicted by estimator and actual recorded values (absolute value)

$$\sqrt{\frac{\sum_{i=1}^{n}(f(x_i)-y_i)^2}{n}}$$

x_i is the speed of a single wheel
y_i is the ratio of peak vertical load to nominal vertical load of a single wheel
$f(x_i)$ is the predicted dynamic factor of a wheel given its speed
n is the total number of wheels

Mean signed difference – summarizes how well an estimator matches the quantity that it is supposed to estimate

$$\sum_{i=1}^{n}\frac{f(x_i)-y_i}{n}$$

x_i is the speed of a single wheel
y_i is the ratio of peak vertical load to nominal vertical load of a single wheel
$f(x_i)$ is the predicted dynamic factor of a wheel given its speed
n is the total number of wheels

Speed-weighted signed difference – signed difference, with weight given for the speed of the wheel

$$\frac{\sum_{i=1}^{n}(x_i f(x_i)-x_i y_i)}{\sum x_i}$$

x_i is the speed of a single wheel
y_i is the ratio of peak vertical load to nominal vertical load of a single wheel
$f(x_i)$ is the predicted dynamic factor of a wheel given its speed
n is the total number of wheels

Load-weighted signed difference – signed difference, with weight given for the nominal wheel load

$$\frac{\sum_{i=1}^{n}(Q_i f(x_i)-Q_i y_i)}{\sum Q_i}$$

Q_i is the nominal load of a single wheel
x_i is the speed of a single wheel
y_i is the ratio of peak vertical load to nominal vertical load of a single wheel
$f(x_i)$ is the predicted dynamic factor of a wheel given its speed
n is the total number of wheels

As shown in Table 5.1, many of the dynamic factors incorporate other parameters. Therefore, several parameters must be held constant to maintain effective comparisons with respect to speed (Table 5.4).

Table 5.4 Parameters held constant for dynamic factor evaluation

Parameter	Constant Value	Justification
Wheel diameter, D	36 in	Typical value for many freight and passenger vehicles in North America
Track modulus, U	6000 psi	Representative of well-maintained concrete-tie track (as found at WILD site)
Track quality, δ	0.1	Representative of track in very good condition (as found at WILD site)
Confidence factor, t	3	Upper confidence limit of 99.7%, applicable for rail stresses, fastenings, and ties

The evaluation was performed using data from three WILD sites (Mansfield, Massachusetts; Hook, Pennsylvania; and Edgewood, Maryland) on Amtrak's Northeast Corridor that experience both higher speed intercity passenger service as well as freight service. After removing the wheels recorded in error (e.g., nominal load of zero) all remaining wheels that traveled over those sites for one month (November 2010) were tabulated and a value for each dynamic factor was calculated based on the speed of the particular wheel and the parameters as found in Table 5.4. Because some of the dynamic factors have ranges in train speed where they are applicable, those values were calculated using only speeds for which that particular dynamic factor is appropriate. The calculated, or expected, dynamic factor was then compared with the ratio of peak vertical wheel load to nominal wheel load using the metrics found in Table 5.3. The results of this comparison are shown in Table 5.5 and graphically in Figures 5.3 through 5.6.

Table 5.5 Evaluation of Dynamic Factors

	Dynamic Factors								
Evaluation Metric	Talbot	Indian Railways	Eisenmann	ORE/ Birmann	German Railways	South African Railways	Clarke	WMATA	Sadeghi
Percent Exceeding	0.234	0.606	0.367	0.753	0.560	0.248	0.454	0.482	0.891
Mean Percentage Error $\frac{100\%}{n}\sum(f(x_i) - y_i)/y_i$	18.408	-7.625	0.229	-11.693	-5.893	15.677	-1.883	-0.383	-15.605
Root Mean Square Deviation $\sqrt{\sum(f(x_i) - y_i)^2 /n}$	0.613	0.528	0.509	0.574	0.558	0.590	0.518	0.572	0.566
Mean Signed Difference $\sum\frac{(f(x_i) - y_i)}{n}$	0.199	-0.186	-0.081	-0.250	-0.164	0.158	-0.101	-0.074	-0.307
Speed-Weighted Signed Difference $\sum(x_i f(x_i) - x_i y_i) / \sum x_i$	0.368	-0.116	-0.031	-0.182	-0.058	0.317	-0.009	0.079	-0.289
Load-Weighted Signed Difference $\sum(Q_i f(x_i) - Q_i y_i)/ \sum Q_i$	0.239	-0.133	-0.018	-0.188	-0.112	0.200	-0.051	-0.027	-0.246

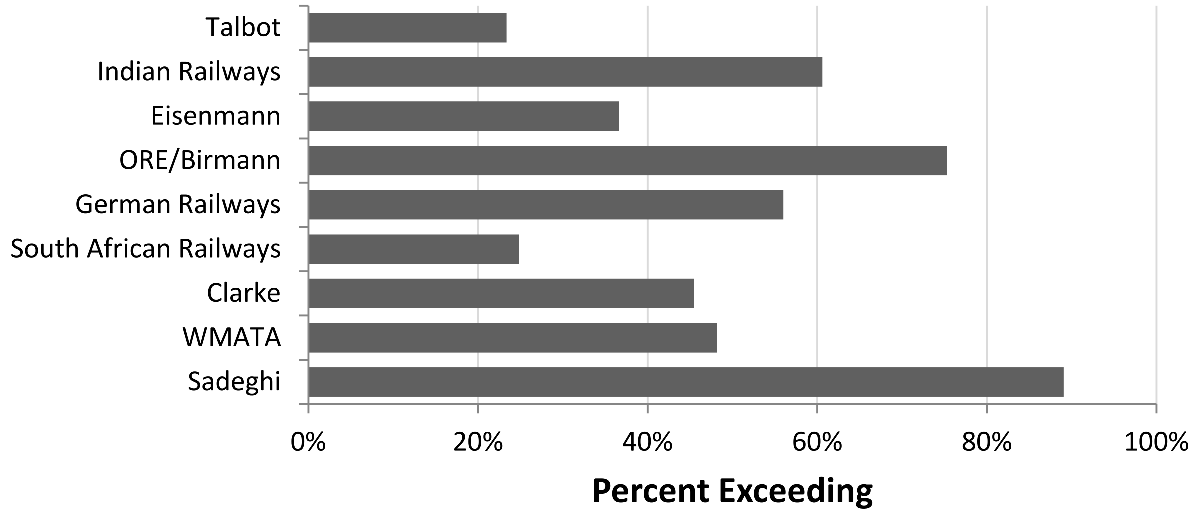

Figure 5.3 Percentage of wheels' peak/nominal ratios exceeding the predicted dynamic factor

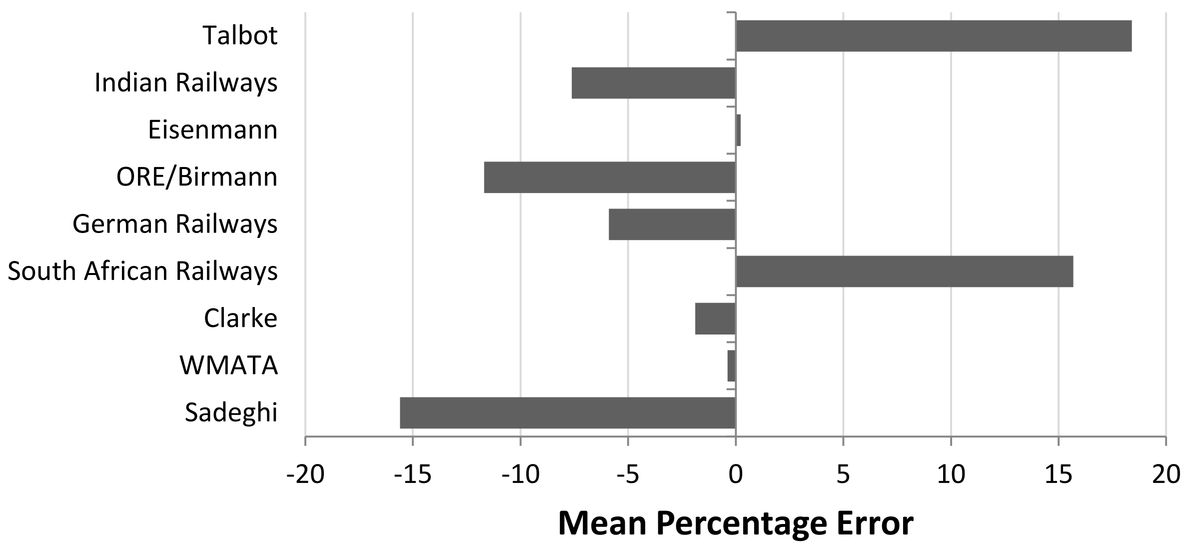

Figure 5.4 Mean percentage error by which predicted dynamic factor differ from peak/nominal ratio

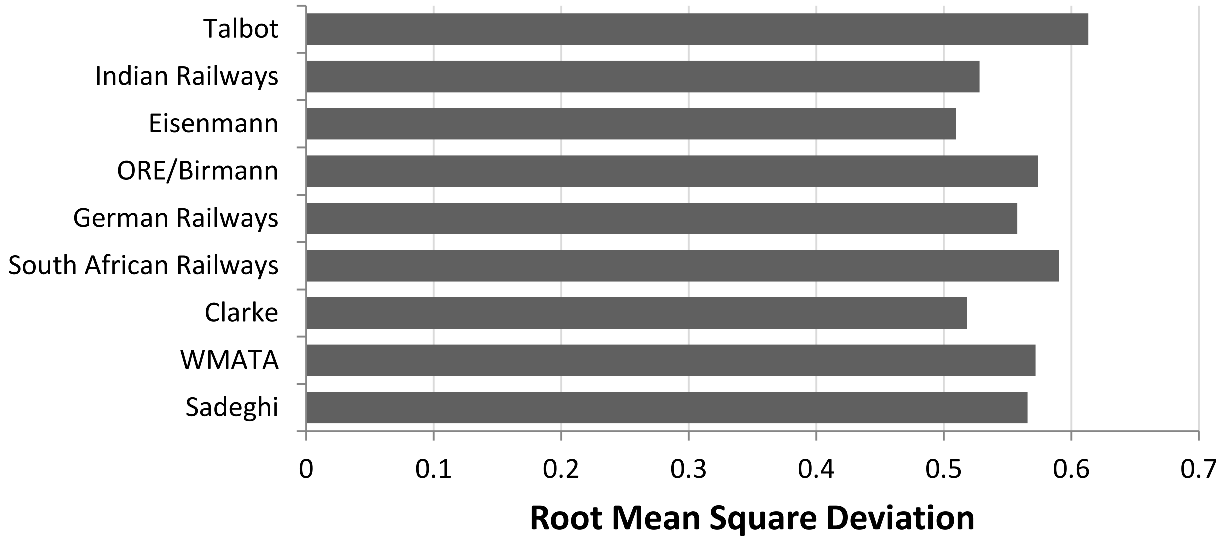

Figure 5.5 Root mean square deviation between predicted dynamic factor and peak/nominal ratio

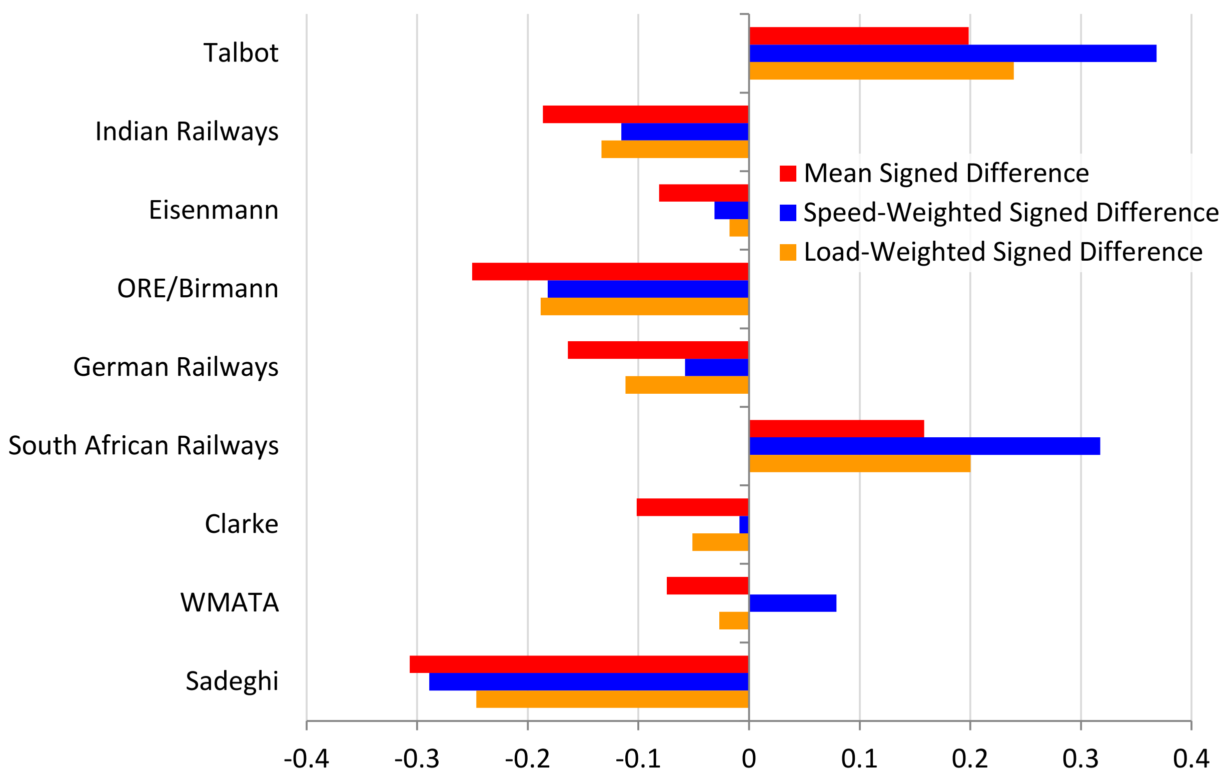

Figure 5.6 Signed differences between predicted dynamic factor and peak/nominal ratio

As is shown in the preceding figures, there are substantial differences between many of the dynamic factors. Using several evaluative metrics, the Eisenmann dynamic factor generally estimates the actual loading well. Positive signed differences, positive mean percentage error, and a low percentage exceedance indicate that the Talbot and South African Railways dynamic factors are fairly conservative when compared to actual loading data. The WMATA speed factor can also be considered conservative by the speed-weighted signed difference metric (likely due to the magnitude of this factor at high speeds, as shown in Figure 5.1). The other dynamic factors are not overly conservative by any of the metrics, but they may still be appropriate in some circumstances.

To better estimate the effect of speed, a linear estimate of wheel load data was developed using WILD data. To isolate the effect of speed, locomotive wheel loads are initially examined for this analysis. In the author's opinion, these wheels are more likely to be more consistently maintained and impart fairly reliable static loads. Therefore, the effect of wheel condition and nominal load can be minimized. The change in dynamic factor due to speed can be expressed as following and is illustrated in Figure 5.7:

$$\frac{Peak}{Nominal} = 1.099 + 0.00386(Speed(mph))$$

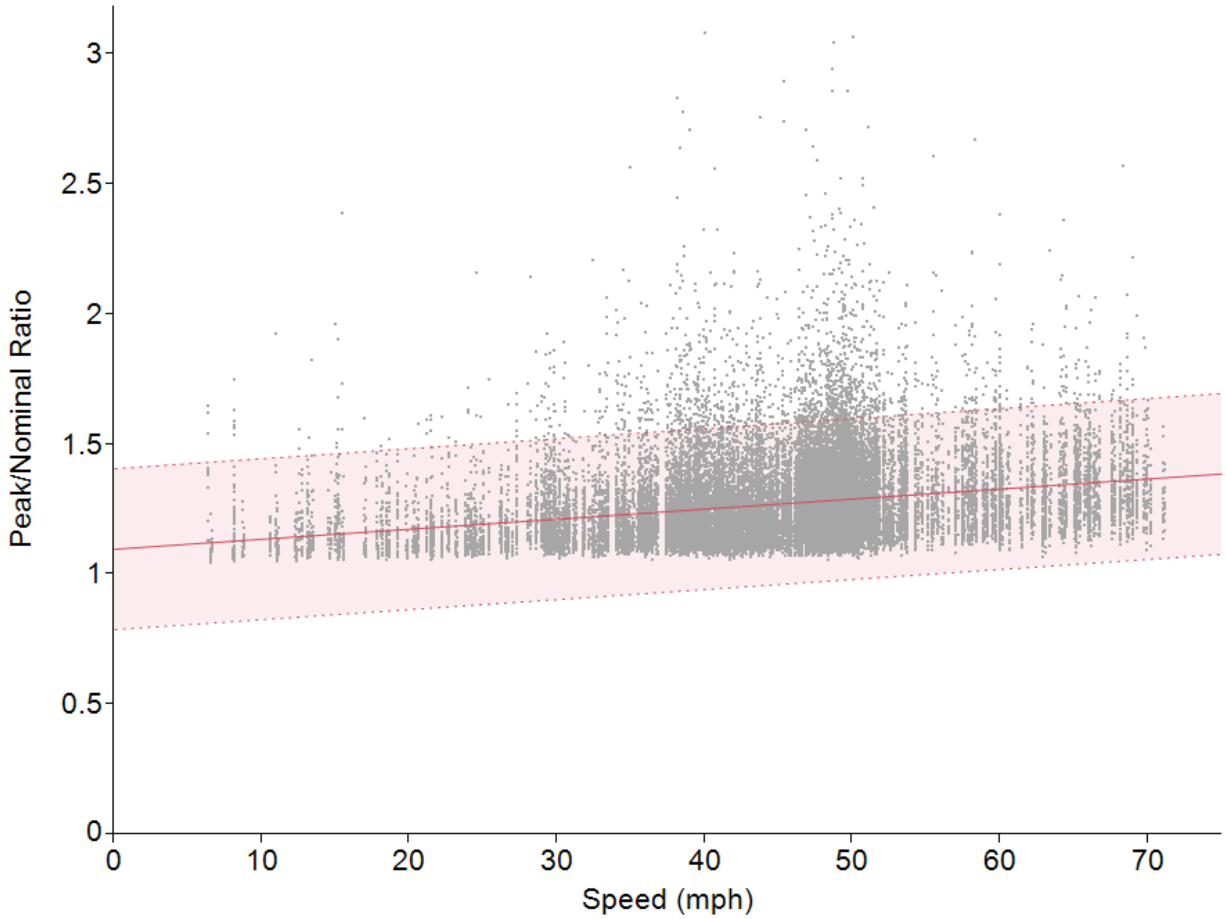

Figure 5.7 Linear Estimate for Dynamic Factor on UPRR at Gothenburg, Nebraska (locomotive WILD data from January 2010)

Figures 5.8 through 5.10 show similar trends for other car types at the same location. Because there is likely more variation due to wheel condition for these car types, the linear estimate may have a greater slope than the effect of speed ought to exhibit. For the purpose of this analysis and any following figures, “unloaded freight cars” include any non-intermodal freight cars whose nominal wheel load is less than 15 kips. The shaded regions represent 95% confidence intervals for the linear estimate. The linear estimates are summarized in Table 5.6.

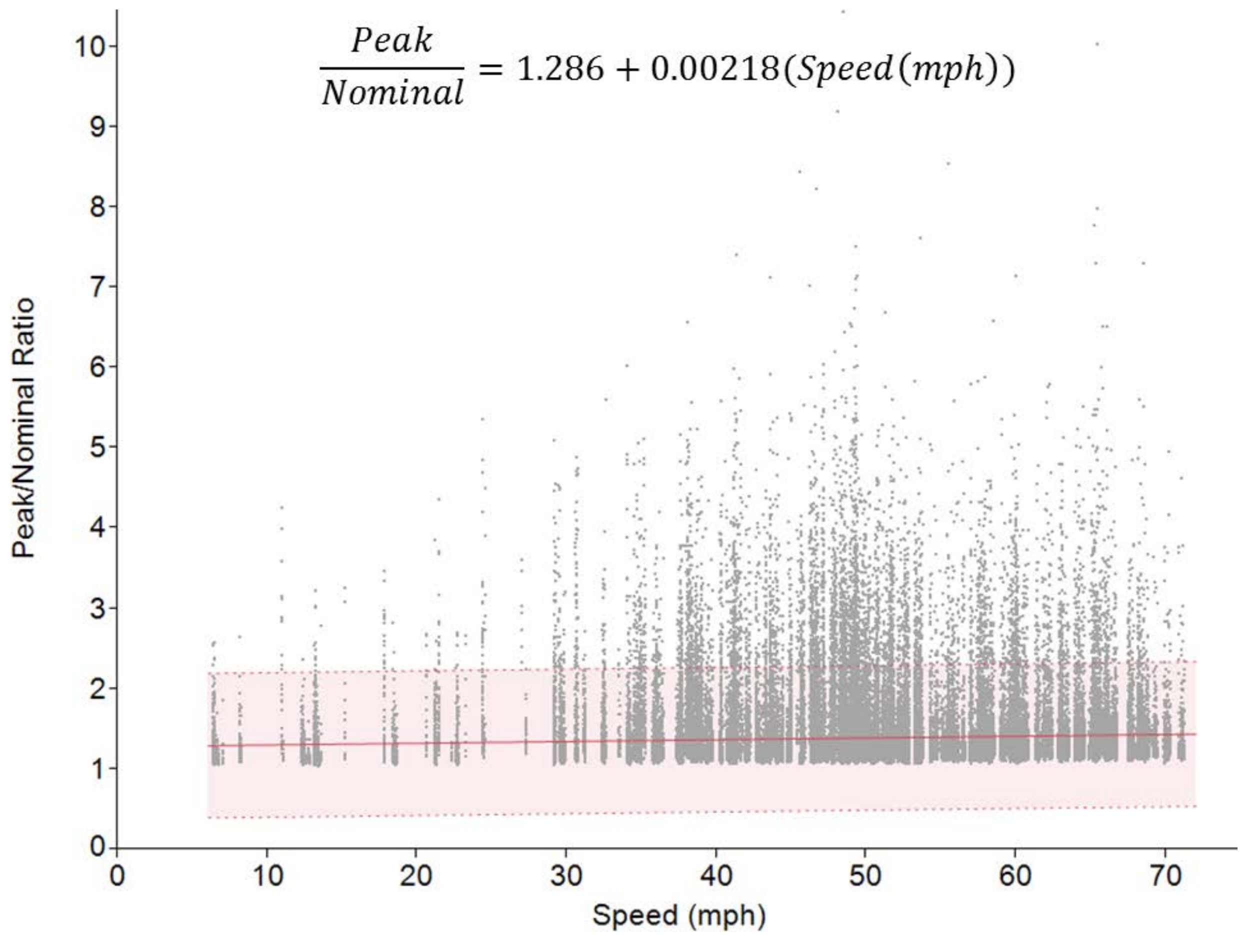

Figure 5.8 Linear Estimate for Dynamic Factor on UPRR at Gothenburg, Nebraska (intermodal freight car WILD data from January 2010)

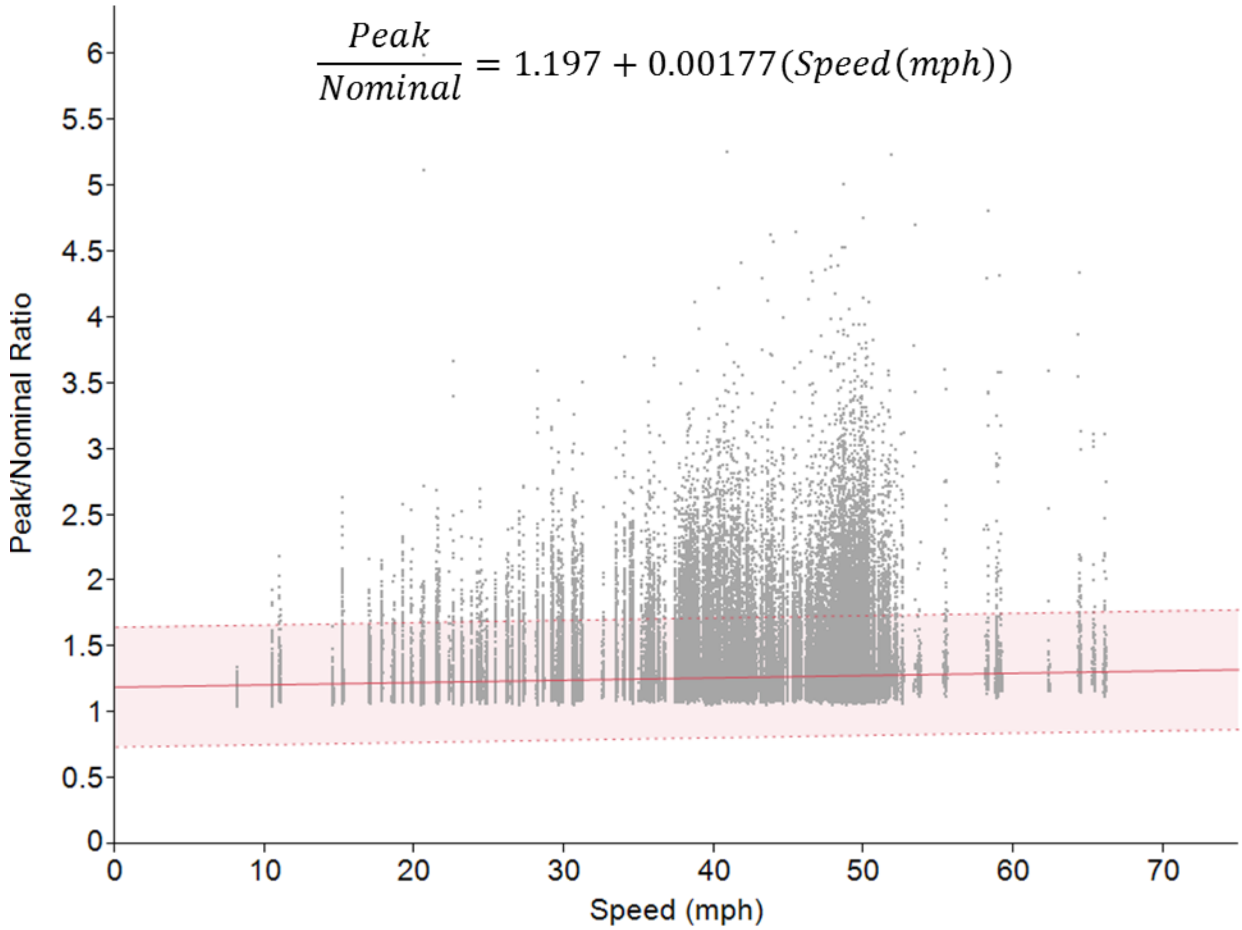

Figure 5.9 Linear Estimate for Dynamic Factor on UPRR at Gothenburg, Nebraska (loaded freight car WILD data from January 2010)

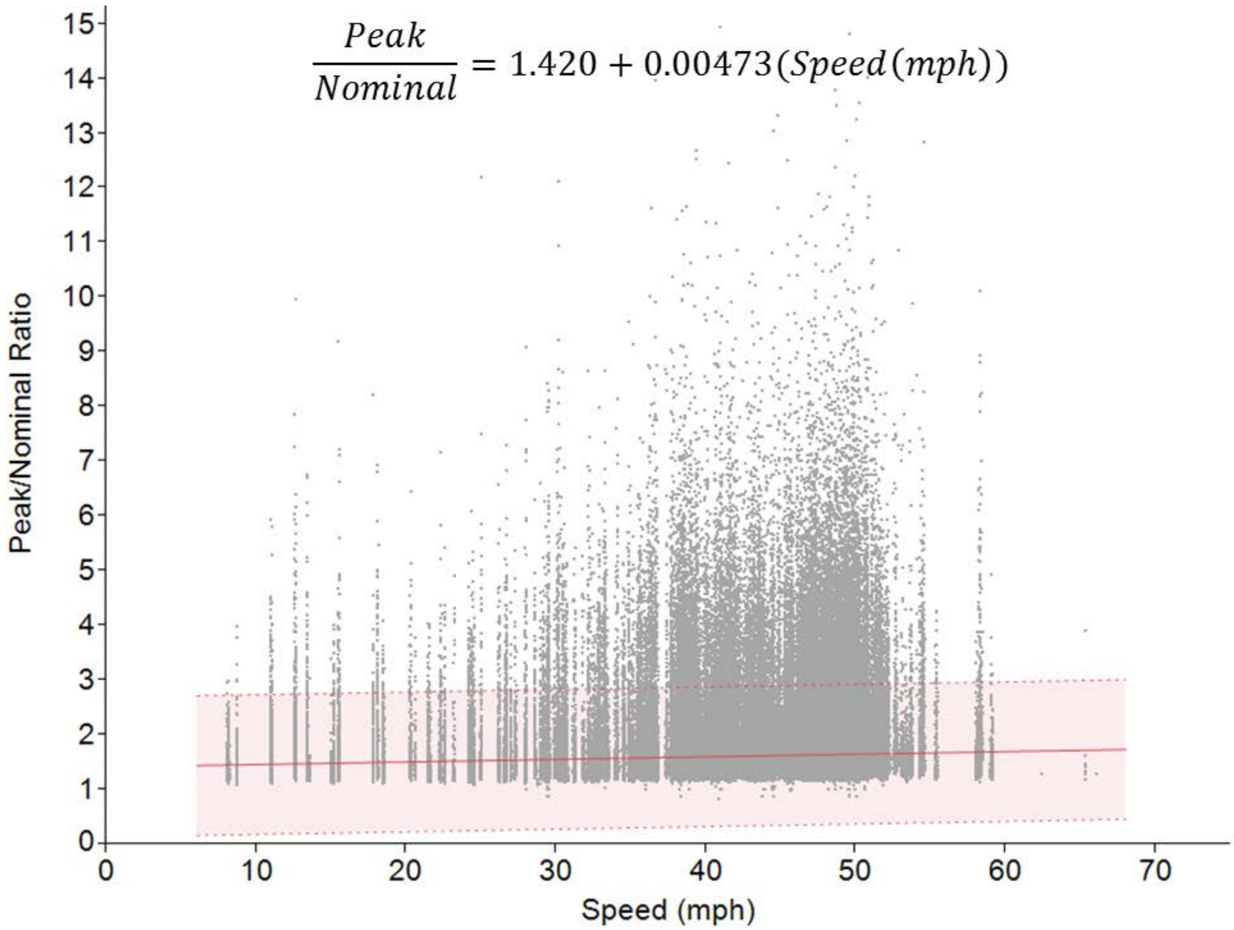

Figure 5.10 Linear Estimate for Dynamic Factor on UPRR at Gothenburg, Nebraska (unloaded freight car WILD data from January 2010)

Table 5.6 Summary of Linear Estimates for Dynamic Factor on UPRR at Gothenburg, Nebraska (WILD data from January 2010)

Car Type	Linear Estimate for Dynamic Factor
Locomotive	$\frac{Peak}{Nominal} = 1.099 + 0.00386(Speed(mph))$
Intermodal Freight Car	$\frac{Peak}{Nominal} = 1.286 + 0.00218(Speed(mph))$
Loaded Freight Car	$\frac{Peak}{Nominal} = 1.197 + 0.00177(Speed(mph))$
Unloaded Freight Car	$\frac{Peak}{Nominal} = 1.420 + 0.00473(Speed(mph))$

While many of the wheel loads do exceed the predicted dynamic factor, it is likely not because of speed. As referenced in Chapter 4, there are other factors that affect the magnitude of wheel load beyond speed. These factors can more appropriately be incorporated into an impact factor.

5.3 Definition and Evaluation of Impact Factor

As shown in Figure 5.2, many wheels create loads much higher than those expected due to speed. Because the dynamic factor does not adequately represent actual loading conditions in terms of impact loads, an additional factor should be utilized. The impact factor is used extensively in bridge design and has been a part of concrete crosstie design since the inception of the American Railway Engineering Association's design recommendations (McQueen 2010).

The AREMA Manual defines the impact factor as a percentage increase over static vertical loads intended to estimate the dynamic effect of wheel and rail irregularities (American Railway Engineering and Maintenance-of-Way Association 2012). An impact factor of 50% was first used, and has incrementally increased to today's 200% level (McQueen 2010). A 200% increase above static load indicates that the design load is three times the static load, hereafter referred to as an impact factor of three. Because the impact factor described in this portion of the recommended practices is specifically related to the flexural performance of the crosstie, it may not be representative of the loads experienced at the wheel-rail interface. Therefore, additional impact factors that may better represent wheel loading conditions shall be explored.

WILD data is again used to evaluate the effectiveness of the AREMA Chapter 30 impact factor (3) and other theoretical impact factors. Figure 5.11 shows actual wheel loading at UPRR's Gothenburg, Nebraska WILD site compared to predicted loads based on various impact factors. Other freight WILD sites yielded similar results, while passenger coach wheels on Amtrak's network exceeded the design impact factors more frequently than those at the freight WILD sites. See Section 4.4 for additional information about variability among WILD sites.

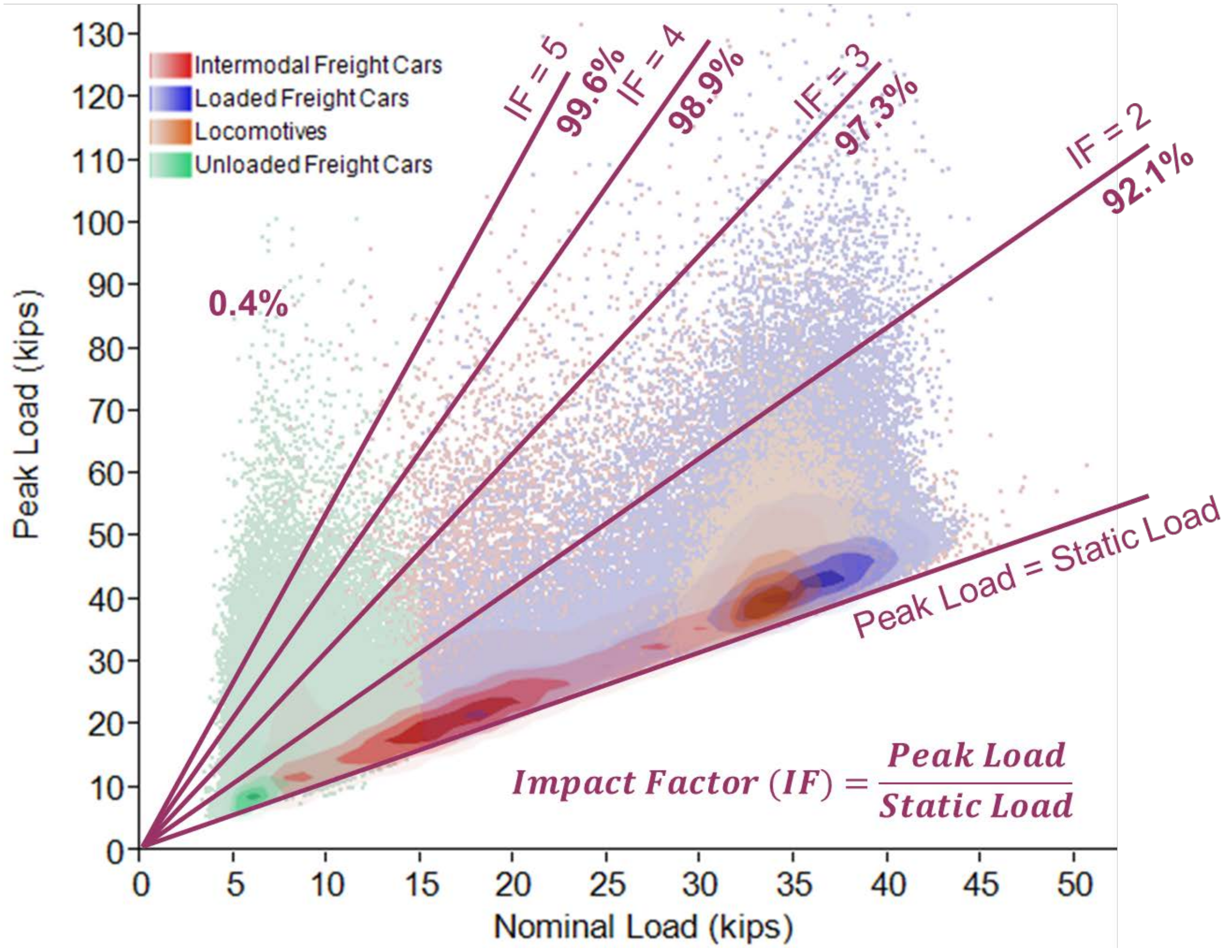

Figure 5.11 Relationship between peak and nominal wheel loads on UPRR at Gothenburg, Nebraska (WILD data from January 2010) and design impact factors

As shown in Figure 5.11, the impact factor of three as found in AREMA Chapter 30 exceeds the majority of the locomotive and loaded freight car loads. Because lighter rolling stock (i.e. passenger coaches and unloaded freight cars) have lower static loads, a higher impact factor can be attained with peak loads similar to those seen with other equipment. Therefore, for these types of vehicles, either a greater impact factor or a different design tool that more effectively represents the full loading spectrum may need to be used.

5.4 Alternative Design Parameters

While dynamic and impact factors have been used for design for close to a century, it is clearly difficult to design based on solely these factors. There is too much variability to be able to cover entire rail networks or even one line with a simple factor. It is, therefore, worthwhile to pursue alternative design parameters to supplement the factors already in use.

5.4.1 Peak Tonnage

Infrastructure owners are typically well aware of the tonnage that traverses each segment of their network. However, this value is calculated by summing the static load of each vehicle, which is not always the best estimate for the actual load entering the track structure (as shown in the previous chapter). Therefore, tonnage that is typically reported, or the "static tonnage", may not necessarily represent true field conditions. By accumulating the peak load of each wheel that passes a WILD site, the "peak tonnage" of a line can be calculated.

Tables 5.7 and 5.8 represent totals at Union Pacific's Gothenburg, Nebraska WILD site. The trends are fairly consistent between years, as shown by the peak-to-nominal wheel load difference per wheel. Table 5.9 shows similar information at UPRR's Sunset, California WILD site, which sees more intermodal traffic.

Table 5.7 Tonnage totals on UPRR at Gothenburg, Nebraska (WILD data from 2010)

Car Type	Number of Wheels	Nominal Tonnage (tons)	Peak Tonnage (tons)	Difference (tons)	Difference per Wheel (tons)
Locomotives	965,718	16,291,645	20,293,696	4,002,051	4.14
Intermodal Freight Cars	3,001,656	28,778,161	38,562,442	9,784,281	3.26
Other Freight Cars	20,204,202	144,556,403	197,330,434	52,774,031	2.61
Total	24,171,576	189,626,209	256,186,572	66,560,363	2.75

Table 5.8 Tonnage totals on UPRR at Gothenburg, Nebraska (WILD data from 2011)

Car Type	Number of Wheels	Nominal Tonnage (tons)	Peak Tonnage (tons)	Difference (tons)	Difference per Wheel (tons)
Locomotives	959,858	16,237,983	20,170,318	3,932,335	4.09
Intermodal Freight Cars	2,651,116	25,353,219	33,885,533	8,532,314	3.22
Other Freight Cars	20,571,408	140,831,724	194,917,926	54,086,202	2.63
Total	24,182,382	182,422,926	248,973,777	66,550,851	2.75

Table 5.9 Tonnage totals on UPRR at Sunset, California (WILD data from 2011)

Car Type	Number of Wheels	Nominal Tonnage (tons)	Peak Tonnage (tons)	Difference (tons)	Difference per Wheel (tons)
Locomotives	165,896	2,793,015	3,437,503	644,488	3.88
Intermodal Freight Cars	749,760	6,133,002	9,017,303	2,884,301	3.85
Other Freight Cars	1,001,596	9,785,716	14,065,909	4,280,193	4.27
Total	1,917,252	18,711,733	26,520,715	7,808,982	4.07

Similar measures can be tabulated on mixed-use lines utilizing data from Amtrak's Northeast Corridor (Tables 5.10 through 5.12). Because the traffic composition and maintenance of rolling stock differs greatly along the corridor, the measurements vary substantially between sites.

Table 5.10 Tonnage totals on Amtrak at Edgewood, Maryland (WILD data from 2011)

Car Type	Number of Wheels	Nominal Tonnage (tons)	Peak Tonnage (tons)	Difference (tons)	Difference per Wheel (tons)
Passenger Locomotives	233,330	3,178,908	4,386,277	1,207,369	5.17
Freight Locomotives	58,452	981,644	1,173,223	191,579	3.28
Passenger Coaches	1,296,790	28,914,644	42,547,772	13,633,128	10.51
Intermodal Freight Cars	237,404	1,683,003	2,254,564	571,561	2.41
Other Freight Cars	1,271,010	12,384,737	17,084,881	4,700,144	3.70
Total	3,096,986	47,142,936	67,446,667	20,303,731	6.56

Table 5.11 Tonnage totals on Amtrak at Hook, Pennsylvania (WILD data from 2011)

Car Type	Number of Wheels	Nominal Tonnage (tons)	Peak Tonnage (tons)	Difference (tons)	Difference per Wheel (tons)
Passenger Locomotives	234,950	2,986,719	3,922,364	935,645	3.98
Freight Locomotives	11,523	186,060	209,773	23,713	2.06
Passenger Coaches	1,529,770	26,040,498	35,181,894	9,141,396	5.98
Intermodal Freight Cars	12,135	119,534	138,446	18,912	1.56
Other Freight Cars	77,746	778,616	938,637	160,021	2.06
Total	1,866,124	30,111,427	40,391,114	10,279,687	5.51

Table 5.12 Tonnage totals on Amtrak at Mansfield, Massachusetts (WILD data from 2011)

Car Type	Number of Wheels	Nominal Tonnage (tons)	Peak Tonnage (tons)	Difference (tons)	Difference per Wheel (tons)
Passenger Locomotives	161,161	2,346,728	3,394,357	1,047,629	6.50
Freight Locomotives	14,304	249,835	303,458	53,623	3.75
Passenger Coaches	831,735	11,856,667	21,325,896	9,469,229	11.38
Intermodal Freight Cars	4,276	34,771	53,171	18,400	4.30
Other Freight Cars	139,953	1,308,788	1,865,539	556,751	3.98
Total	1,151,429	15,796,789	26,942,421	11,145,632	9.68

Design processes that involve tonnage may be able to take advantage of existing peak tonnage values and apply them to other segments with similar traffic composition. Those that are more axle-load-oriented may be able to use the appropriate “difference per wheel” value in addition to the expected static loads on a particular line. This measurement helps to provide an accurate increase of load, but it does not address the particular reasons for increase.

It should be noted that the peak tonnage measurement is not a completely accurate representation of actual tonnage either. Because the values are attained using “peak” loads over a discrete length of track (16 crosstie cribs (GeMeiner 2005)), the majority of the track structure may not experience loads at such a high magnitude. However, the quantities are also measured at well-maintained WILD sites, eliminating any track-related increase in loads. Therefore, the peak tonnage may provide an adequate estimation of actual tonnage.

5.4.2 "Risk"

It is well understood that a measure of risk can be calculated using some product of frequency and severity (National Transportation Safety Board 1971). If applied to the track structure, this concept can involve the frequency of wheel passes and the severity (i.e. peak load) of each wheel pass.

Figure 5.12 shows a typical probability distribution for peak wheel loads at a WILD site on Amtrak's Northeast Corridor. Peak vertical load is used as a proxy for severity, and is shown on the x-axis. Frequency, or the number of wheels, is shown on the y-axis. If these values are multiplied, each data point can represent a "risk" at that particular load. Figure 5.13 illustrates levels of risk at particular levels of peak vertical wheel load.

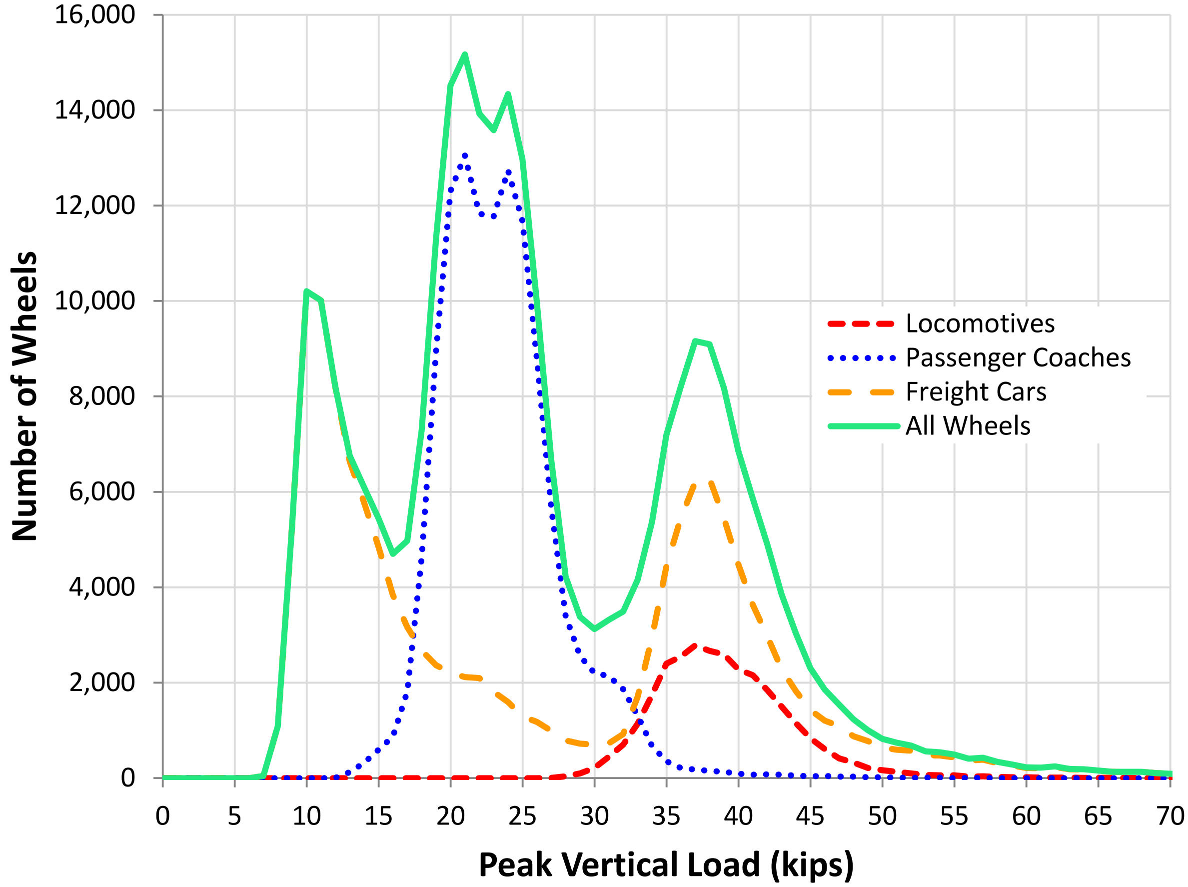

Figure 5.12 Frequency of wheels by peak vertical load on Amtrak at Edgewood, Maryland (WILD data from November 2010)

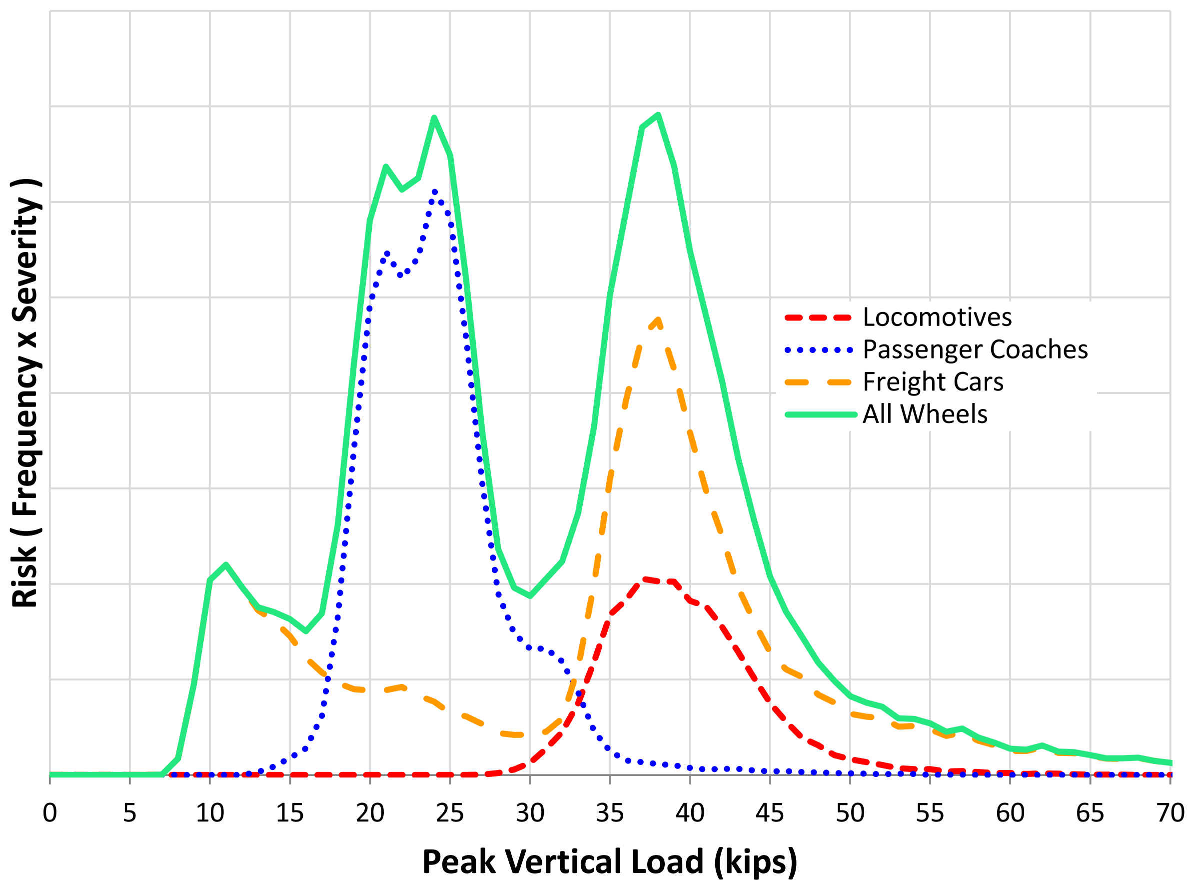

Figure 5.13 Risk of particular peak vertical loads on Amtrak at Edgewood, Maryland (WILD data from November 2010)

Figure 5.13 shows that the combination of severity and frequency of the higher peak loads (represented by primarily freight cars and locomotives) provides a nearly equivalent risk to that of the lower peak loads (represented by mostly passenger coaches (Figure 5.12)). This risk concept could therefore be used to design for a specific type of car imparting a known peak load if the frequency of that car type is well-established.

5.5 Conclusions

There have been many efforts to quantify the effect of speed and irregularities in the form of dynamic and impact factors, respectively. As shown in this chapter, some represent today's loading environment in North America better than others. Depending on the metric used to evaluate each factor, the factors vary in their conservatism. The appropriate level of design should be selected by the infrastructure owner, and more than one factor may be necessary in determining the design wheel load for the track infrastructure. Higher-degree estimates and dynamic factors that include other parameters may be developed and evaluated in the future to better represent the dynamic wheel loading environment. Rigorous statistical methods may be used to effectively model the effect of speed and many other factors.

Two additional design parameter methodologies have been proposed, providing additional information that was not necessarily evident with the dynamic and impact factors. Multiple factors may be needed to adequately represent the existing wheel loads on the North American rail network and improve design of the critical components that make up the track structure.

CHAPTER 6: CONCLUSIONS AND FUTURE RESEARCH

6.1 Conclusions

The primary objective of this thesis was to characterize the loading environment of shared-use infrastructure to improve the design of critical superstructure components, especially concrete crossties and elastic fastening systems. To achieve this objective, many strategies were employed and will be explored in the future. The following sections provide an overview of the conclusions related to the major topics found in this thesis.

6.1.1 International Concrete Crosstie and Fastening System Survey Conclusions

According to the survey responses, the axle loads and tonnages on concrete crosstie territory in North America are, on average, much higher than those found throughout the rest of the world. On average, both North American and international maximum freight loads exceed the design load for crossties. Many of the failure modes experienced in North America are related to wear or fatigue surrounding the rail seat area, and, subsequently, the most significant research thrusts moving forward are related to reducing deterioration at the rail seat, shoulder, and rail pad assembly interfaces.

The most important international research thrusts are track design and optimization, which has yielded many innovative design methodologies. Because several types of crosstie cracking are prevalent forms of failure internationally, many of the innovative design methodologies may address a probabilistic view of system loading and support. More varied crosstie manufacturing techniques internationally may also contribute to substantially different trends in requirements and performance of concrete crossties.

According to the responses, the elastic fastening systems are designed while considering the track system as a whole. Component and system interaction plays a large role in the fastening system design process and ought to be considered in the mechanistic design of the track structure.

6.1.2 Overview of Loading Environment Variation in North America

The WILD has been shown to be a useful tool for collecting and analyzing data related to loads entering the track structure. Using this tool, the following conclusions can be made regarding wheel loads in North America:

- Vehicle type and its associated static load provides a baseline for the expected total load at the wheel-rail interface
- Increasing speed minimally increases the most common wheel loads; however, severe impact loads become more severe at higher speeds
- Traffic composition and other site-specific parameters play a substantial role in the distribution of the loading environment
- Seasonal effects in load variation, while greatly contributing to the magnitude of severe impacts, minimally affect the majority of the wheel load distribution
- Wheel condition is a substantial factor in determining peak loads entering the track structure
- Lateral loads on both rails increase with increased cant deficiency on curved track

The application of these conclusions, in addition to the relationships between parameters that incorporate other data collection methods, will contribute to a greater understanding of overall wheel load variation, leading to improved design of the track structure.

6.1.3 Evaluating Dynamic and Impact Factors

The effects of speed and wheel-rail irregularities are considered in design through the use of dynamic and impact factors. Many of these factors were developed using wheel loads that are no longer representative of today's rail networks. A thorough evaluation is, therefore, necessary to determine the appropriateness of each factor for particular design processes. The results of this evaluation could lead to improved design factors and potentially innovative design methodologies that ultimately lead to better performance of critical infrastructure components.

6.2 Future Work

The work described in this thesis can be used as a foundation for future research with this and additional datasets. The following sections will describe the use of WILD and other data in the future to contribute to improved design of track superstructure components.

6.2.1 Developing Improved Mechanistic Design Practices

A primary objective of the UIUC Federal Railroad Administration (FRA) Tie and Fastener Broad Agency Announcement (BAA) research program is to develop improved mechanistic recommended design practices for shared-use superstructure components, particularly concrete crossties and elastic fastening systems. While Chapter 3 provided a summary of UIUC's vision for the mechanistic design process, there is still substantial work to be done to deliver improved recommended practices. The work found in this thesis will contribute to the mechanistic design practice, especially as it pertains to system input loads.

The load is then traced throughout the remainder of the track structure, using established scientific principles related to stress and strain. Each component is evaluated using its materials and geometry by comparing its load-carrying capacity to the expected load passing through it. As described in Chapter 3, extensive laboratory and field testing is used as validation for a comprehensive finite element model. The model will be able to change properties of the track structure while monitoring stress and strain behavior within each component given certain loading protocols, predicting failure and other improvement areas. This process will lead to improved design of critical infrastructure components, increasing life cycles and safety on North American rail networks.

6.2.2 Further Analyzing WILD Data

WILD data have been used extensively in the analysis presented in this thesis. While the dataset does have some limitations (described in Chapters 4 and 5), it has been proven to be useful in investigative definitions of particular trends and relationships related to wheel loads. However, there are still many applications for this particular dataset that have not been explored.

Some statistical analysis has been performed thus far on these data, but it merely provides a framework for more rigorous statistical methods and testing. Due to the size of the dataset (nearly 89 million records in one year for one railroad), it lends itself very well to the applications of descriptive statistics, which provide more straightforward conclusions without making inferences. There are many unused parameters within the data that may be used to provide greater insight to the loading environment and other areas of research within railroad engineering.

Ultimately, the WILD data can be used to develop a loading environment model. Using existing data to develop relationships between speed and wheel load, quantitative trends can be established and used to predict peak wheel loads (Figure 6.1). Irrespective of vehicle speed, the nominal, or static, wheel load can be used as a proxy for expected peak wheel load given a particular car type (Figure 6.2). A regression model could predict an expected load given a particular set of parameters related to additional traffic and track characteristics, many of which are discussed at a high level in Chapter 4. This model could then be validated using additional data and modified according to different operating scenarios, resulting in further refinement of the loading environment characterization at the wheel-rail interface.

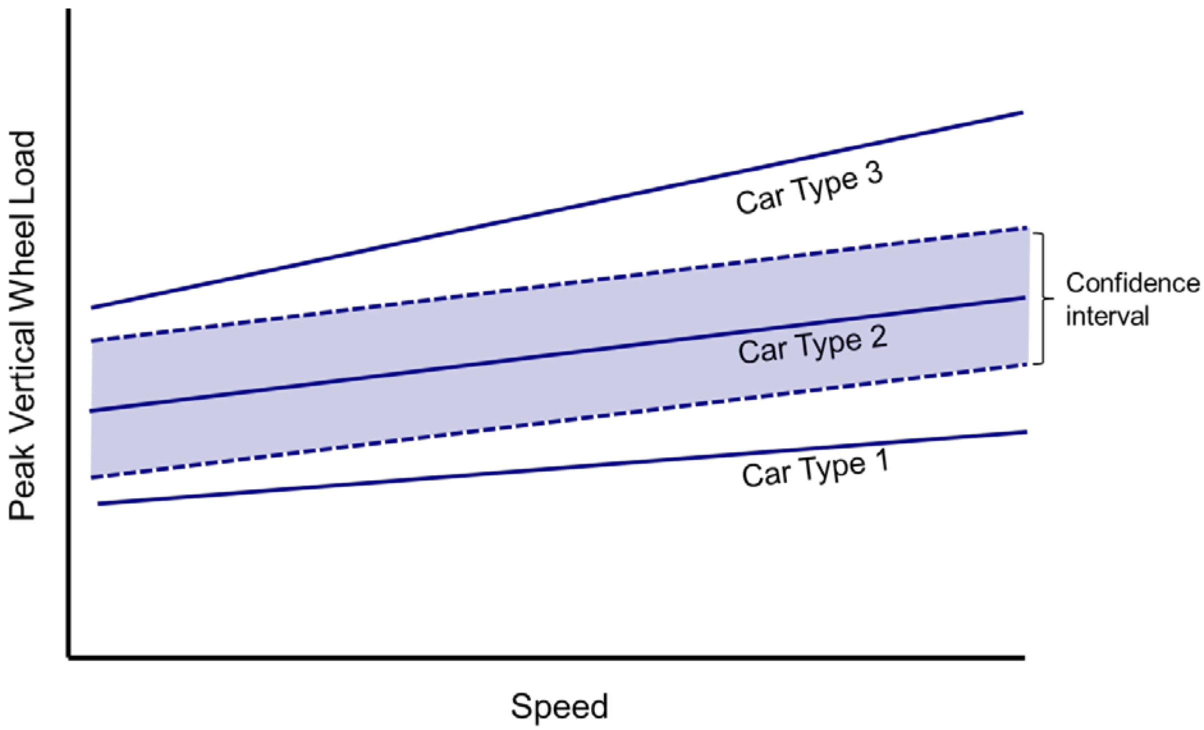

Figure 6.1 Conceptual application for speed-related loading environment model using WILD data

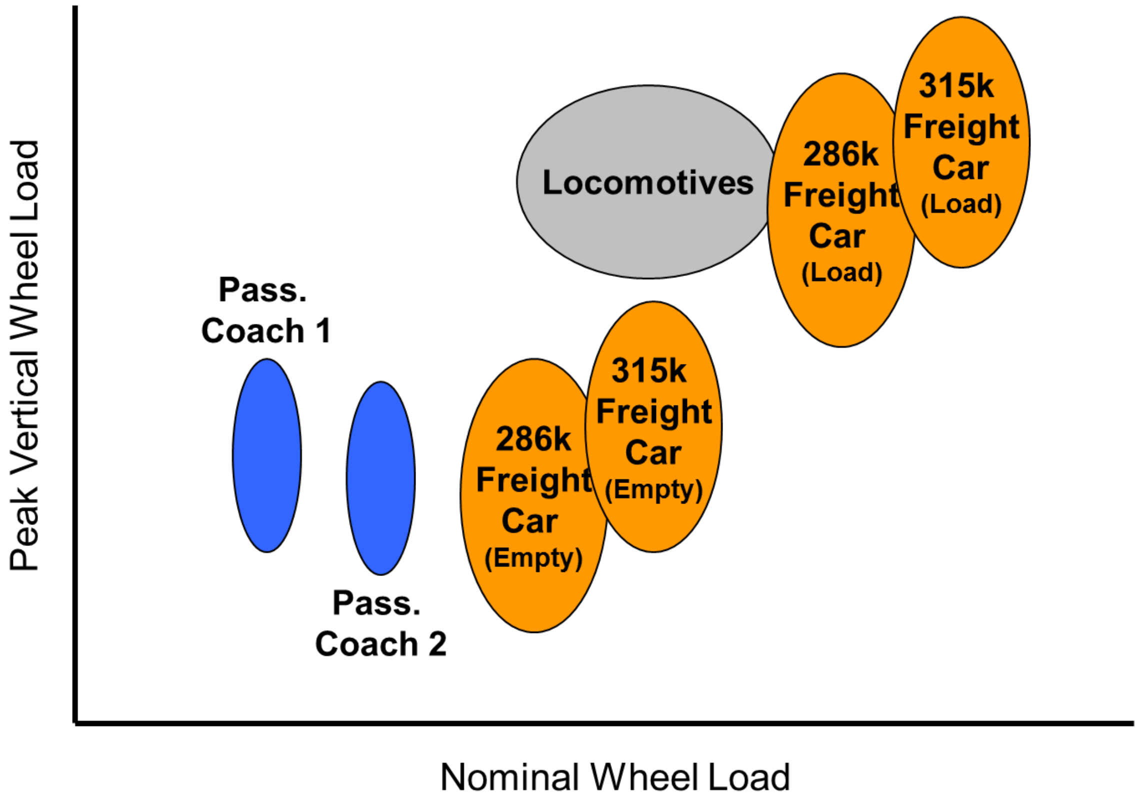

Figure 6.2 Conceptual application for loading environment model using WILD data

6.2.3 Quantifying Loads Using Additional Technologies

As discussed in Chapter 4, WILD data does not paint a complete picture of the loading environment at the wheel-rail interface. While the WILD does collect lateral loading information, it does not experience substantial lateral load magnitudes because it is located on tangent track. Additionally, the WILD does not provide any information related to longitudinal wheel loads. Vertical loads have historically been the focus of many design methodologies, but the lateral and longitudinal loads cause many failures throughout the track structure. Therefore, in the future, lateral and longitudinal loads will be investigated using alternative data collection technologies, such as the instrumented wheel set (IWS), truck performance detector (TPD), and the UIUC Instrumentation Plan.

IWS analysis was briefly discussed in Chapter 4 in terms of lateral loads and cant deficiency, but this technology can provide extensive information related to wheel-rail interaction and how vertical, lateral, and longitudinal loads vary throughout the duration of curves and curve transitions. Because many failure modes occur exclusively in curved track, this technology can provide useful information relating to wheel loads that may be causing these failures.

The TPD and the UIUC Instrumentation Plan (Edwards et al. 2014) are both wayside measurement technologies that provide information related to wheel loads in tangent and curved track. The data collected from these systems can be used to compliment WILD data with wheel loads in curved sections of track. The UIUC Instrumentation Plan can provide insight throughout the track superstructure as well. Ultimately all of these technologies can be used collectively to better characterize the loading environment experienced by North America’s track structure.

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APPENDIX A: INFRASTRUCTURE OWNER, OPERATOR, OR MAINTAINER RESPONSES

3. Please identify a representative route within your network that best fits the following criteria:- Mainline with higher than average tonnage- Concrete sleepers and elastic fastening systems in place for at least fifteen years- High curvature and grade relative to the rest of the network- In general, offers demanding operating conditionsHereafter, this route will be referred to as the "typical route".

	8 Responses

4. Freight Train Loading

	Question 4: What are the maximum gross static wheel loads?	Question 4: What is the typical dynamic load impact factor? (%) (e.g. 200% = 2 x static loading)
INTL	24.8 tons (22.5 tonnes)	not of concern
	24.8 tons (22.5 tonnes)	
	18.7 tons (17.0 tonnes)	Velocity(km/h)×0.5/100
	38.6 tons (35.0 tonnes)	200
	29.2 tons (26.5 tonnes)	200
	30.3 tons (27.5 tonnes)	250
US	44.0 tons (39.9 tonnes)	150
	18.0 tons (16.3 tonnes)	?
	17.9 tons (16.2 tonnes)	220

5. Passenger Train Loading

	Question 5: What are the maximum gross static wheel loads?	Question 5: What is the typical dynamic load impact factor? (%) (e.g. 200% = 2 x static loading)
INTL	24.8 tons (22.5 tonnes)	not of concern
	22.0 tons (20.0 tonnes)	
	13.2 tons (12.0 tonnes)	Velocity(km/h)×0.5/100
	33.1 tons (30.0 tonnes)	200
	22.6 tons (20.5 tonnes)	204
US	N/A	
	12.5 tons (11.3 tonnes)	180

6. What is the average speed of trains?

	30-60 miles per hour (50-100 kilometers per hour)	5	56%
	60-90 miles per hour (100-150 kilometers per hour)	2	22%
	90-120 miles per hour (150-200 kilometers per hour)	1	11%
	120-150 miles per hour (200-250 kilometers per hour)	0	0%
	150-180 miles per hour (250-300 kilometers per hour)	0	0%
	Other, please specify	1	11%
	Total	9	100%
US	90-150 mph passenger, 30-50 mph freight		

7. Please provide the following axle spacings.				
	Question 7: What is the minimum axle spacing on freight wagons?	Question 7: What is the average axle spacing on freight wagons? (i.e. length of most common wagon divided by number of axles)	Question 7: What is the minimum axle spacing on passenger carriages?	Question 7: What is the average axle spacing on passenger carriages? (i.e. length of most common carriage divided by number of axles)
INTL	5.9 feet (1.8 meters)		6.6 feet (2.0 meters)	26.4 feet (8.0 meters)
	26.4 feet (8.0 meters) for bogie wagons, 32.8 feet (10.0 meters) for axle wagons	55.8 feet (17.0 meters)	50.5 feet (15.4 meters) between bogies and 6.9 feet (2.10 meters) between axles	
	5.2 feet (1.6 meters)	6.2 feet (1.9 meters)	6.9 feet (2.1 meters)	6.9 feet (2.1 meters)
	I do not know	I do not know	I do not know	I do not know
	32.5 feet (9.9 meters)	39.3 feet (12.0 meters)		
	5.2 feet (1.6 meters)	6.2 feet (1.9 meters)	7.9 feet (2.4 meters)	8.2 feet (2.5 meters)
US	Unknown	Unknown	Unknown	Unknown
	Standard freight and coal equipment			
		10 feet (3.0 meters)		10 feet (3.0 meters)

8. Is locomotive sand used on your network to increase wheel adhesion and prevent wheels from slipping?

Yes	9	100%
No	0	0%
Total	9	100%

9. What is the annual tonnage per track?	
INTL	2.2 million tons (2.0 million tonnes)
	3.9 million tons (3.5 million tonnes)
	22.0 million tons (20.0 million tonnes)
	88.2 million tons (80.0 million tonnes)
	71.7 million tons (65.0 million tonnes)
	33.1 - 55.1 million tons (30 - 50 million tonnes)
US	varies widely from 50.0 - 250.0 million tons (45.4 - 226.8 million tonnes)
	50.0 million tons (45.4 million tonnes)
	10.0 - 45.0 million tons (9.1 - 44.8 million tonnes)

Sleepers

10. Please provide the typical sleeper spacing for the following track segments.			
	Question 10: Tangent	Question 10: Curve	Question 10: Grade Crossing
INTL	23.6 inches (60.0 centimeters)	23.6 inches (60.0 centimeters)	23.6 inches (60.0 centimeters)
	23.6 inches (60.0 centimeters)	23.6 inches (60.0 centimeters)	
	23.6 inches (60.0 centimeters)	22.8 inches (58.0 centimeters)	22.8 inches (58.0 centimeters)
	24.0 inches (61.0 centimeters)	24.0 inches (61.0 centimeters)	24.0 inches (61.0 centimeters)
	27.0 inches (68.5 centimeters)	27.0 inches (68.5 centimeters)	27.0 inches (68.5 centimeters)
	23.6 inches (60.0 centimeters)	23.6 inches (60.0 centimeters)	19.7 - 23.6 inches (50.0 - 60.0 centimeters)
US	24.0 inches (61.0 centimeters)	24.0 inches (61.0 centimeters)	18.0 - 24.0 inches (45.7 - 61.0 centimeters)
	24.0 inches (61.0 centimeters)	24.0 inches (61.0 centimeters)	
	24.0 inches (61.0 centimeters)	24.0 inches (61.0 centimeters)	24.0 inches (61.0 centimeters)

11. What is the typical area of your rail seat?	
INTL	23.3 square inches (150.0 square centimeters)
	44.6 square inches (288.0 square centimeters)
	46.5 square inches (300.0 square centimeters)
	1020.0 square inches (6580.0 square centimeters)
	40.3 square inches (260.0 square centimeters)
	44.6 square inches (288.0 square centimeters)
US	54.6 square inches (352.4 square centimeters)
	standard
	29.5 square inches (190.3 square centimeters)

12. What is the specified rail seat inclination (referred to as cant in North America)? (e.g. 1:40)	
INTL	1:40
	1:20
	1:40
	1:40
	1:20
	1:20
US	1:30 (pre 2007), 1:40 (post 2007)
	1:40
	1:40

13. Which companies and facilities manufacture the sleepers on your typical route? (manufacturer, city, and country of facility)	
INTL	Local Swiss concrete suppliers
	SATEBA, France
	There are many manufacturers
	ROCLA Concrete Tie, Denver, CO, USA
	Austrak, Rockhamption, Australia
	ROCLA, Bowral, NSW Australia
US	CXT, Grand Island/Tucson/Spokane, USA; Rocla, Amarilo, USA; NorTrak, Cheyenne, USA
	KSA, Sciotoville, OH, USA
	Rocla, Bear, DE, US

Fastening Systems

14. Fastening System Trends

	Question 14: Historically, what types of fasteners have been most commonly used? (brand and model, e.g. Pandrol e-CLIP)	Question 14: Currently, what types of fasteners are most commonly installed? (brand and model, e.g. Pandrol e-CLIP)	Question 14: If these two answers are different, please explain the design and performance advantages of the system that is currently installed.
INTL	Vossloh K12 etc	Several different Vossloh types, depending on sleeper.	
	NABLA System	NABLA System	
	According to Japan Industrial Standard (JIS)	JIS Type 5 (tangent) or Type 9 (curved)	
	e-clip 78-late 80s Safelok 87-2008 Vossloh 2008 and current	Safelok has the largest population- about 10 Million ties.	Evolution changes: Clip fatigue drove the change from e-clip to Safelok. Shoulder and insulator wear drove the change from Safelok to Vossloh.
	Pandrol e-clip	Pandrol e-clip	
	Pandrol e-Clip	Pandrol e-Clip	
US	In order of quantity: Pandrol Safelok III Pandrol Salelok I Pandrol e-clip Vossloh	Pandrol Safelok III	The vast majority of fasteners installed on concrete ties on our territory are Pandrol Safelok III. This remains our standard as the fastener provides improved toe loads versus all previous Pandrol products. The Vossloh fastening system is currently under test.
	e Fast Clip	Fast Clip	Captive Fast Clip design for initial installation.
	Pandrol fast clip, Pandrol e-clip	Pandrol fast clip	Ease of installation of fast clip

15. What is the fastener clamping force (toe load)?

INTL	2248 pounds force (10.0 kilonewtons)
	According to track structure (ballasted/slab)
	4496 pounds force (20.0 kilonewtons)
	6774 pounds force (30.0 kilonewtons)
	2360 pounds force (10.5 kilonewtons) per clip
US	2500 - 2900 pounds force (11.1 - 12.9 kilonewtons)
	2250 pounds force (10.0 kilonewtons)

16. What is the rail pad material?

	Polyurethane	6	67%
	Rubber	2	22%
	Other, please specify	1	11%
	Total	9	100%
US	HDPE		

17. What is the rail pad geometry?			
	Dimpled	2	25%
	Grooved	0	0%
	Studded	1	13%
	Flat	1	13%
	Other, please specify	4	50%
	Total	8	100%
INTL	Ribbed		
	Dimpled and corrugated can yield the same results. We use both.		
US	Proprietary info		
	All four pad styles are presented.		

18. What is the material of the component in the fastening system that provides electrical insulation?	
INTL	polyamid
	plastic
	polyurethane tie pad and nylon insulator or angle guide plate plus plastic insert
	polyurethane
	HDPE
US	polyurethane & nylon
	polyurethane & nylon
	nylon

19. Is a frame or plate used between the rail pad and sleeper?			
	Yes	3	43%
	No	4	57%
	Total	7	100%

20. If so, from what material is it constructed?	
INTL	steel
	We are still testing frames vs conventional gasket, steel plate + tie pad
US	plastic or steel

21. How many years have concrete sleepers and fastening systems been used by your railroad?	
INTL	90
	about 60
	34
	30
	25 - 35
US	22
	28
	34

Effectiveness

22. Concrete Sleeper Life

	Question 22: What is the design life of the concrete sleepers? (years)	Question 22: What percentage of your concrete sleepers remain in service beyond their design life?	Question 22: Of the concrete sleepers that do not achieve their design life, what is their average service life?	Question 22: What is the most common reason for replacing concrete sleepers prior to achieving their design life?
INTL	35 estimated for these old sleepers (no design life fixed)	10%		fastening system disorders
	25	0%		Defect of screw
	30	less than 1 %. After all only 10 miles have been in track that long.	Not known.	shoulder wear.
	50	not there yet	don't know yet	fist fastener sleepers
	50	0%	3 years	impact force (from various sources); severe sleeper or rail seat abrasion
US	50 years is the desired tie life, with the maximum actual tie life currently at 22 years	We have not reached the desired tie life on any of our ties.	5-10 years	bond loss
	?	0%	15 years	failed
	50	haven't reached design life yet	unkown	failure due to mechanical breakage or ASR

23. Fastening System Life

	Question 23: What is the design life of the fastening systems? (years)	Question 23: What percentage of your fastening systems remain in service beyond their design life?	Question 23: Of the fastening systems that do not achieve their design life, what is their average service life?	Question 23: What is the most common reason for replacing fastening systems prior to achieving their design life?
INTL	> 30	?	?	fastening system disorders (anchoring)
	25	Large lateral force	Defect of spring	
	life of the rail	None		insulator wear
	30 so far	n/a	25	fist fastener sleepers due to corrosion of pin
	50	10	n/a	damage, unfit, electrical resistance
US	Not measured in years, but in tonnage which is 1.2 BGT (high curvature) - 3 BGT (tangent)	Unknown	Loss of toe load	Capital project rail change outs
	Life of rail	0	5	failed or wide gage
	same as tie life			mechanical breakage

24. Do you perform any maintenance (replacement, repair, etc.) on your concrete sleepers and fastening systems?

Yes	8	100%
No	0	0%
Total	8	100%

31. Please rank the following concrete sleeper and fastening system problems on your network from most to least critical.

Top number is the count of respondents selecting the option. Bottom % is percent of the total respondents selecting the option.

	1	2	3	4	5	6	7	8
Derailment damage	0 0%	0 0%	3 43%	0 0%	4 57%	0 0%	0 0%	0 0%
Cracking from center binding	1 17%	1 17%	0 0%	3 50%	1 17%	0 0%	0 0%	0 0%
Cracking from dynamic loads	1 17%	1 17%	1 17%	1 17%	1 17%	1 17%	0 0%	0 0%
Cracking from environmental or chemical degradation	0 0%	0 0%	0 0%	1 17%	0 0%	1 17%	3 50%	1 17%
Deterioration of concrete material beneath the rail	1 14%	2 29%	0 0%	1 14%	0 0%	1 14%	1 14%	1 14%
Shoulder/fastening system w ear or fatigue	3 43%	1 14%	1 14%	0 0%	0 0%	2 29%	0 0%	0 0%
Tamping damage	0 0%	2 29%	2 29%	0 0%	1 14%	1 14%	1 14%	0 0%
Other (e.g. manufactured defect)	2 33%	0 0%	0 0%	1 17%	0 0%	1 17%	0 0%	2 33%

32. Of the following potential failure causes, please select any and all that have resulted in deficiencies of your concrete sleepers and fastening systems.

	Deficient concrete strength	3	60%
	Improper prestress force	2	40%
	Poor material quality or behavior (of clamp, insulator, rail pad, or sleeper)	5	100%
	Poor environmental conditions (e.g. moisture or fines intrusion)	1	20%
	Manufacturing flaw s	5	100%
	Improper component design (of clamp, insulator, rail pad, or sleeper)	5	100%
	Fastening system damage	3	60%
	Concrete deterioration beneath the rail	4	80%
	Poor bonding of concrete to prestress	3	60%
	Other, please specify	2	40%
INTL	Ranking orger: Insulator loads exceed capacity w hich can result in shoulder w ear		
US	ASR		

Practices

33. What set of standards or industry-recommended practices do you follow for the design, manufacture, testing, and installation of concrete sleepers and fastening systems?

INTL	Euro Norms + Internal standards
	according to Japanese Industrial Standard
	Internal standards considering AREMA and Euro-Norm
	australian standards
	RailCorp Standards/Specifications and Australian Standard
US	many
	AREMA
	Internal specifications, AREMA, ASTM

34. What types of tests do you execute on concrete sleepers and fastening systems? Please refer to specific sections in the standard stated in the previous answer, when applicable.

INTL	Euro Norms + Internal standards
	according to Japanese Industrial Standard
	We have a long list of concrete tie specifications.
	visual inspection and concrete testing of compressive strength
	Many tests as per RailCorp Specifications and Australian Standards
US	many from ASTM, ACI, PCI
	none except mfg. required by AREMA
	refer to Amtrak Concrete Tie specification

35. What additional general comments do you have on concrete sleeper and fastening system design, manufacture, testing, and installation?

INTL	Complex problem. We believe that we have a pretty good structural tie design. We are ALWAYS looking for improvements. The fastening area have the most opportunities for improvement. WE want the fastener and rail life to match without maintenances!
	sleepers need reduced thickness
	High speed rails require a proper design of fastening system. Urban rails and Frieghts require a very good maintenance of rail system.
US	make stronger field shoulder; avoid sharp curves or decrease spacing
	We need to continue research. We can do better. We need to better understand the dynamic loading environment, how the tie responds to these loads and how we can improve our testing procedures to better match what the ties will see in the field.

Research

36. In your opinion, what are the most important topics of research regarding concrete sleepers and fastening systems? Please rank the following areas of concrete sleeper and fastening system research from most to least beneficial.

Top number is the count of respondents selecting the option. Bottom % is percent of the total respondents selecting the option.

	1	**2**	**3**	**4**	**5**
fastening systems design: clamps, insulators, inserts, rail pads	3 38%	4 50%	0 0%	1 13%	0 0%
materials design: concrete mix, prestress strand arrangement	1 13%	1 13%	1 13%	4 50%	1 13%
optimize sleeper design: spacing, cross-section, body shape, for specific uses (curves, grades, etc.)	0 0%	1 13%	5 63%	2 25%	0 0%
prevention of concrete deterioration under the rail or repair of abraded sleepers	1 13%	1 13%	1 13%	1 13%	4 50%
track system design: determining the track service environment and required sleeper characteristics	3 38%	1 13%	1 13%	0 0%	3 38%

37. Has concrete sleeper and fastening system research been performed by your railroad or other parties on your sleepers and fastening systems?

Yes	8	100%
No	0	0%
Total	8	100%

38. If so, on what primary topics has research been conducted?

INTL	Life cycle (cost and remaining strength)
	ladder type sleeper
	RSD
	toe loads
	impact loading, strength and serviceability, design concept, reliability and safety, noise & vibration, railseat abrasion, void and pocket, dynamic characteristics, integrated sensors, etc.
US	concrete tie life cycle, fastener life cycle, pad life cycle, rail seat repair, etc.
	lateral loads
	premature failures

39. Please provide references to literature published by your railroad or by outside parties on your railroad.

INTL	There are many papers. Please search the author "Hajime WAKUI".
	Private.
	nil
	Published data available in http://www.ro.uow.edu.au Internal data (+100 tech reports) has been internally available (also available to our academic researchers via RailCRC). Not available to public.
US	TTCI, otherwise all other research is witheld
	N/A

40. If unpublished test results have been documented regarding the research conducted by your railroad, would you be willing to share relevant information with the University of Illinois at Urbana-Champaign research team?

Yes	5	63%
No	3	38%
Total	8	100%

190. Please enter the following general information. Any information obtained on this page will remain confidential and will not be released.

8 Responses

191. Please briefly describe the technical responsibilities related to your position.

8 Responses

192. If you are aware of any other individuals who would be able to offer relevant information, please provide their names and e-mail addresses.

4 Responses

193. What proprietary restrictions exist with the information you have provided in this survey?

7 Responses

APPENDIX B: ACADEMIC, INDUSTRY, OR INSTITUTIONAL RESEARCHER RESPONSES

41. Concentration of Research

	Question 41: What are your specific areas of research? (e.g. infrastructure components, subgrade, structures)	Question 41: Specifically, how are you involved with concrete sleepers and fastening systems? (e.g. instrumenting sleepers, modeling of fastening systems)
INTL	Infrastructure components, stiffness, actions, fastenings, sleepers	Modeling of: track, fastenings, sleepers. Sleepers' testing. To propose a reliable method for calculating the actions on track.
	Studying Master of Engineering (Rail Infrastructure) at QUT	Current course unit UDN500 Ballast & Sleepers
	track structures and components incluing fastening, sleeper and concrete slab	modeling and analysis, experiment and on-site testing on sleeper and fastening systems
	Concrete railway sleepers and bridges. Our university track research group is dealing also with all the other components of railway track (subsoil, subballast, ballast, rail, wheel-rail interaction)	general research on concrete sleepers
	infrastructure components and systems	theoretical design, modelling, component tests, field measurements
	Materials for especially concrete sleepers, subgrade improvement	development of new eco-friendly PC sleeper
	Railway track mechanic and dynamic infrastructure engineering	Field research on sleepers and CWR, lab research on fastening systems and rail joints
	concrete sleepers and railway track dynamics	experimental and numerical investigation of sleepers
	track degradation and component life, track dynamics, track stiffness, track modelling, wheel-rail forces	limit states design and rating of concrete sleepers, static and fatigue testing of sleepers, sleeper life modelling, study of impact forces on sleepers
US	track structure	modeling, insrumenting and testing of cross ties

Based on your expertise as described in the previous answers, please answer the following questions to the best of your ability as they apply in your country.If railroads in your country have different types of concrete sleepers and fastening systems in their networks, please respond to this survey based on the sleeper and fastening system most commonly used in demanding operating conditions.

42. What operating conditions would you consider to be demanding?

INTL	Mixed traffic passenger and freight in High-Speed lines (Vmax=200-250 km/h), axle-load 22.5 t/axle
	Heavy haul traffic, High speed passenger traffic
	Freight trains with flat wheels running on the same track as passenger trains
	High Speed, Heavy Haul
	conventional railway under the speed of 200km/hr
	durability of concrete sleepers, optimizing dimensions, life cycle, reliability analysis, vibration noise absorption
	High speed 120-150 km/h, high axle loads
	heavy axle loads, dirty environment (dust or sand from the ground or from mineral payloads, borne by air or water), poor maintenance of the rail head or of the wheel treads, high speed trains
US	315k lbs cars, sharp curves, hilly and/or rainy areas.
	Mainline coal routes, mountainous terrain

43. Freight Train Loading

	Question 43: What are the maximum gross static wheel loads?	Question 43: What is the typical dynamic load impact factor? (%) (e.g. 200% = 2 x static loading)
INTL	24.8 tons (22.5 tonnes)	Depending on the case it maybe arrive 3 times the static load and if there is fault on the rail's running surface even higher
	16.5 tons (15 tonnes)	Unknown
	12.1 tons (11.0 tonnes)	250%
	13.8 tons with 62 mile per hour speed (12.5 tonnes with 100 kilometers per hour speed)	highly dependent on flat wheels, but for a sleeper typically maybe around 150-200%
	27.6 tons (25.0 tonnes)	150%
	44.1 tons (40 tonnes)	200%
	24.3 tons (22.0 tonnes)	130-150%
	27.6 tons (25.0 tonnes)	133%
	35.3 tons (32.0 tonnes)	200-250%
	22.0 tons (20.0 tonnes) per wheel for heavy axle wagons containing coal or iron ore; 14.3 tons (13.0 tonnes) per wheel for ordinary freight	250%
US	19.5 tons (17.7 tonnes)	150%
	41 tons (37.2 tonnes)	

44. Passenger Train Loading

	Question 44: What are the maximum gross static wheel loads?	Question 44: What is the typical dynamic load impact factor? (%) (e.g. 200% = 2 x static loading)
INTL	24.8 tons (22.5 tonnes)	as in freight, a little bit less perhaps
	8.7 tons (7.9 tonnes)	Unknown
	12.1 tons (11 tonnes) for conventional lines and 13.8 tons (12.5 tonnes) for high-speed lines in design (but actual wheel load of Korean high speed train is 9.4 tons (8.5 tonnes))	200% for ballasted and 150% for slab track
	8.8 tons with 137 miles per hour speed (8.0 tonnes with 220 kilometers per hour speed)	highly dependent on flat wheels, but for a sleeper typically maybe around 150-200%
	24.8 tons (22.5 tonnes)	150%
	28.7 tons (26.0 tonnes)	150%
	18.7 tons (17.0 tonnes)	150-160%
	27.6 tons (25.0 tonnes)	133%
	N/A	N/A
	12.1 tons (11 tonnes) per wheel	250%
US	20 tons (18.1 tonnes) (light rail)	

45. What is the maximum allowable speed under such demanding operating conditions?

30-60 miles per hour (50-100 kilometers per hour)	3	25%
60-90 miles per hour (100-150 kilometers per hour)	0	0%
90-120 miles per hour (150-200 kilometers per hour)	4	33%
120-150 miles per hour (200-250 kilometers per hour)	1	8%
150-180 miles per hour (250-300 kilometers per hour)	2	17%
Other, please specify	2	17%
Total	12	100%

INTL	(100-105 mph (160-170 kph) in track designed for operational 120-150 mph (200-250 kph))
	Passenger: 60-120 mph (100-200 kph); Freight: 30-60 mph (50-100 kph)

46. Please provide the typical sleeper spacing for the following track segments.			
	Question 46: Tangent	Question 46: Curve	Question 46: Grade Crossing
INTL	23.6 inches (60.0 centimeters)	23.6 inches (60.0 centimeters)	23.6 inches (60.0 centimeters)
	24.0 inches (61.0 centimeters)	24.0 inches (61.0 centimeters)	24.0 inches (61.0 centimeters)
	24.6 inches (62.5 centimeters) for ballasted track and 25.6 inches (65.0 centimeters) for slab track	same	same
	24.0 inches (61.0 centimeters)	24.0 inches (61.0 centimeters)	24.0 inches (61.0 centimeters)
	23.6 inches (60.0 centimeters)	23.6 inches (60.0 centimeters)	23.6 inches (60.0 centimeters)
	23.6 inches (60.0 centimeters)	23.6 inches (60.0 centimeters)	23.6 inches (60.0 centimeters)
	24.6 inches (62.5 centimeters)		
	23.6 inches (60.0 centimeters)		
	23.6-24.4 inches (60.0-62.0 centimeters)		
	23.6-27.6 inches (60.0-70.0 centimeters)	23.6-27.6 inches (60.0-70.0 centimeters)	23.6-27.6 inches (60.0-70.0 centimeters)
US	24.0 inches (61.0 centimeters)	24.0 inches (61.0 centimeters)	24.0 inches (61.0 centimeters)
	24.0 inches (61.0 centimeters)	24.0 inches (61.0 centimeters)	24.0 inches (61.0 centimeters)

47. What are the five (5) most common concrete sleeper designs used in your country? (manufacturer and sleeper identification) (e.g. RAIL.ONE NS 90)	
INTL	Twin-block U2, Twin-block U3, Twin-block U31 (all of them French design and Greek production meeting absolutely the pre-scriptions) patent and license agreement and know-how transfer SATEBA Monoblock pre-stressed B70 (German design and Greek production meeting absolutely the pre-scriptions) patent and license agreement and know-how transfer in three factories: Dywidag, Pfleiderer (now RAILONE), Walterbau. Monoblock pre-stressed for metric gauge line license Moll (German)
	Austrak
	KNR 60kg rail PC sleeper(Korean standards) High speed railway sleeper(Korean standards) Rail.One concrete sleeper for Rheda2000 track
	Only 3 new designs available (2 Finnish manufacturers): Parma BP 99, Parma BP 89 (minor amount) and Luja B97
	Two kinds of Korean sleepers (50kg N and 60kg K) are manufactured by Taemyung industry, Samsung industry, Is dongseo, Jeail con, Sampyo.
	B 70 B58
	AUSTRAK and ROCLA
	Austrak and Rocla are the two main manufacturers and their most common size of heavy duty sleepers are 22cm deep, 20-25cm wide, and 250cm long. Both companies also manufacture low profile sleepers around 17cm deep, 20-22cm wide, and 250cm long.
US	CXT, Rocla, Koppers
	Rocla

48. What are the five (5) most common fastening system designs used in your country? (manufacturer and fastening system identification) (e.g. Vossloh W 14 HH)	
INTL	RN, Nabla designer French company STEDEF W14 German company Vossloh Gmbh
	Pandrol e clip Pandrol Fastclip Fist BTR
	Pandrol e-Clip Pandrol SFC with FC 1501 Vossloh System 300
	Vossloh W 14, Pandrol E-CLIP for replacement of old similar fasteners.
	Vossloh W3, Vossloh W14, Vossloh System 300,
	Railtech Fastclip - e clip Vossloh W21 - W14
	Pandrol products are common in Korea for the conventional line under the speed of 150km/hr Several products are installed for the high-speed line
	Vossloh Pandrol SKL 12
	Pandrol e-clip and Pandrol fast-clip
	Pandrol, Vossloh, e-clip, fastclip, fistclip
US	Saflok I and III e-clips Vossloh
	Vossloh 101L Safelok 101L

49. How many years have concrete sleepers been used in your country?	
INTL	since 1972
	30 plus
	about 40
	monoblock sleepers from year 1964 (at first a German type)
	> 55
	25
	about 50
	20
	40
	40
US	30-40
	+/- 35

Effectiveness

50. What is the most common cause of early replacement of concrete sleepers in your country?	
INTL	Not sufficient strength, not correct design
	Derailment damage
	flexural failure due to unsupported condition and longitudinal cracking
	In general, the need for early replacement has not been significant. Frost weathering. Transversal cracks in sleepers.
	chemical influences
	Derailment
	to increase its weight for track's stability
	longitudinal cracks inside sleepers, cracks under the sleeper due to durability problems
	cracking in rail seat zone
	Derailment damage
US	Generally concrete ties do not complete service life cycle. They are replaced after the lessons are learnt.
	Cracking and spalling

51. What is the most common cause of early replacement of fastening systems in your country?	
INTL	Not correct toe-load, not correct design, high value of static stiffness meaning high value of actions on track
	Fastener corrosion
	broken clip and early hardening of railpad
	Loose fastening
	elasticity
	Clip breakage
	noise and vibration
	failure
	fatigue
US	broken fasteners
	Broken fasteners

52. Have railroads in your country ever experienced the type of deterioration in the sleeper as shown in the images below?

Yes	4	36%
No	7	64%
Total	11	100%

53. If so, what term would you use to identify this deterioration?

INTL	RSD
	Rail Seat Abrasion
	wear - indentation
	to need to estimate the train loading in order to design the sleeper
US	rail seat abrasion

54. Please briefly describe the characteristics of this deterioration, in terms of where it occurred, at what rate it occurred, to what depth it occurred, etc.

INTL	I cannot see clearly, could you please send me more clear and detailed photos?
	depth: 0.02 to 0.04 inches (0.5 to 1.0 millimeters)
	Although rail seat abrasion is perceived to be a big problem in the USA, it's relatively rare in Australia despite many 1000s of kilometres of concrete sleepered track, and it generally occurs only in very dirty environments; abrasion of the underside of the sleeper (due to tamping damage and abrasion from ballast forces due to heavy axle load traffic) is far more common and over a period of 30 years up to 2cm can be lost that way.
US	In general, US railroads have this problem. I do not have direct exposure to this issue.

55. Please rank the following concrete sleeper and fastening system problems in your country from most to least critical.

Top number is the count of respondents selecting the option. Bottom % is percent of the total respondents selecting the option.

	1	2	3	4	5	6	7	8
Derailment damage	2 18%	0 0%	1 9%	1 9%	1 9%	3 27%	2 18%	1 9%
Cracking from center binding	0 0%	3 38%	2 25%	1 13%	0 0%	2 25%	0 0%	0 0%
Cracking from dynamic loads	2 18%	2 18%	3 27%	1 9%	1 9%	1 9%	1 9%	0 0%
Cracking from environmental or chemical degradation	4 44%	0 0%	1 11%	1 11%	2 22%	0 0%	1 11%	0 0%
Deterioration of concrete material beneath the rail	0 0%	1 11%	1 11%	0 0%	0 0%	2 22%	1 11%	4 44%
Shoulder/fastening system wear or fatigue	0 0%	2 22%	0 0%	4 44%	3 33%	0 0%	0 0%	0 0%
Tamping damage	2 20%	2 20%	1 10%	2 20%	1 10%	1 10%	1 10%	0 0%
Other (e.g. manufactured defect)	1 11%	1 11%	2 22%	0 0%	1 11%	0 0%	2 22%	2 22%

56. Of the following potential failure causes, please select any and all that have resulted in deficiencies of concrete sleepers and fastening systems in your country.

Deficient concrete strength	1	9%
Improper prestress force	0	0%
Poor material quality or behavior (of clamp, insulator, rail pad, or sleeper)	4	36%
Poor environmental conditions (e.g. moisture or fines intrusion)	5	45%
Manufacturing flaws	4	36%
Improper component design (of clamp, insulator, rail pad, or sleeper)	3	27%
Fastening system damage	5	45%
Concrete deterioration beneath the rail	3	27%
Poor bonding of concrete to prestress	2	18%
Other, please specify	4	36%

INTL	poor bonding of concrete to reinforcement rods in "normal" concrete twin-block sleepers
	corrosion of fasteners and attrition of concrete from underside of sleeper due to pumping track
	insufficient support from ballast/embankment
	longitudinal cracks on the surface of sleeper

Research

57. In your opinion, what are the most important topics of research regarding concrete sleepers and fastening systems? Please rank the following areas of concrete sleeper and fastening system research from most to least beneficial.

Top number is the count of respondents selecting the option. Bottom % is percent of the total respondents selecting the option.

	1	2	3	4	5
fastening system design: clamps, insulators, inserts, rail pads	2 18%	1 9%	4 36%	4 36%	0 0%
materials design: concrete mix, prestress strand arrangement	0 0%	1 10%	3 30%	4 40%	2 20%
optimize sleeper design: spacing, cross-section, body shape, for specific uses (curves, grades, etc.)	4 36%	5 45%	2 18%	0 0%	0 0%
prevention of concrete deterioration under the rail or repair of abraded sleepers	1 11%	1 11%	1 11%	1 11%	5 56%
track system design: determining the track service environment and required sleeper characteristics	4 40%	3 30%	1 10%	1 10%	1 10%

58. Has concrete sleeper and fastening system research been performed by your organization?

Yes	10	83%
No	2	17%
Total	12	100%

59. If so, on what primary topics has research been conducted?

INTL	stiffness, toe-load, actions on track, life-cycle, compatibility of clip and pad
	deisgn and perfomance verification of rail fastening design
	Field tests, several types of tests. Loading tests at our university, static and cyclic. Structural calculations. Literature review. Interviews.
	Sleeper design, Fastenings Elasticity,
	loads and deflection
	to design new fastening system and sleeper for high-speed railway and the reduction of noise and vibration
	durability of concrete, sleeper optimization, reliability analysis and design of sleeper and fastening systems
	resistance of concrete sleepers to severe impact loads
	these topics were all laid out in my responses at the start of this survey
US	primary focus is to reduce the life cycle cost.

60. Please provide references to literature published by your organization regarding concrete sleepers and fastening systems.

9 Responses - available upon request

61. If unpublished test results have been documented regarding the research conducted by your organization, would you be willing to share relevant information with the University of Illinois at Urbana-Champaign research team?

Yes	9	90%
No	1	10%
Total	10	100%

190. Please enter the following general information. Any information obtained on this page will remain confidential and will not be released.

12 Responses

191. Please briefly describe the technical responsibilities related to your position.

12 Responses

192. If you are aware of any other individuals who would be able to offer relevant information, please provide their names and e-mail addresses.

4 Responses

193. What proprietary restrictions exist with the information you have provided in this survey?

6 Responses

APPENDIX C: CONCRETE CROSSTIE MANUFACTURER RESPONSES

If your organization manufactures different types of sleepers, please respond to this survey based on the most commonly-used sleeper for primary lines, hereafter referred to as the "typical sleeper".

62. What is your typical sleeper? (manufacturer and sleeper identification) (e.g. RAIL.ONE NS 90)

7 Responses

Concrete

63. What is the concrete design mix?

5 Responses

64. What is the design air content of the concrete mix? (% or range of %)

INTL	confidential
	1.0 - 1.3 %
	4.50%
US	5.50%
	3 - 6 %
	3 - 5 %

65. What type of cement is used? (e.g. Type III cement)

INTL	confidential
	CEM II/A-S42,5R WT38
	high early strength (in spec)
US	Type III low alkali
	fine grind type II
	TYPE III
	Type II LA

66. What type of aggregate is used?

Limestone	1	14%
Dolomite	0	0%
Granite	2	29%
Basalt	0	0%
Other, please specify	4	57%
Total	7	100%

INTL	confidential
	Moraine gravel, crushed (limestone-rich)
	river rock, traditionally; now from foot of mountains

67. What is the shape of the aggregate?

Rounded	0	0%
Crushed	6	100%
Total	6	100%

68. What is the average slump of your concrete at placement?

INTL	confidential
	not applicable C0
	4.7 inches (120 millimeters)
US	9.0 inches (229 millimeters)
	7.0 inches (178 millimeters)
	3.0 inches (76 millimeters)

69. What consolidation method is used?

	Vibration mechanism	5	71%
	Self-consolidating concrete	1	14%
	Physical compaction of concrete	0	0%
	Other, please specify	1	14%
	Total	7	100%
INTL	confidential		

70. What methods are used to control concrete curing? Please select all that apply.

	Curing membrane (e.g. wet burlap)	3	43%
	Liquid curing compound	0	0%
	Steam	3	43%
	None	0	0%
	Other, please specify	5	71%
INTL	confidential		
	water basin under air-tight curing stack		
US	oil		
	Radiant Heat		

71. What is the maximum allowable internal temperature of the typical sleeper during curing?

INTL	confidential
	113 °F (45 °C)
	122 - 140 °F (50 - 60 °C)
US	140 °F (60 °C)
	158 °F (70 °C)
	140 °F (60 °C)
	140 °F (60 °C)

72. What is the minimum allowable concrete strength at prestress transfer?

INTL	confidential
	7000 pounds per square inch (48 megapascals)
	6000 pounds per square inch (41 megapascals)
US	5000 pounds per square inch (34 megapascals)
	5000 pounds per square inch (34 megapascals)
	4200 pounds per square inch (29 megapascals)
	4500 pounds per square inch (31 megapascals)

73. What is the average time that elapses between concrete placement and transfer of prestress forces to the concrete? (hours)

INTL	confidential
	36
	17 (17-24 hours for turning beds; 1 per day)
US	8.25
	17
	8 - 14

74. Is the surface of the rail seat treated in any way?		
Yes	2	40%
No	3	60%
Total	5	100%

75. If so, how is it treated? (e.g. polished, added polyurethane, etc.)	
INTL	confidential
US	epoxy
	Approx 50% of ties are epoxy railseats

76. What is the design 28-day compressive strength of your concrete mix?			
	3 - 4.5 kips per square inch (20-30 megapascals)	0	0%
	4.5 - 6 kips per square inch (30-40 megapascals)	0	0%
	6 - 7.5 kips per square inch (40-50 megapascals)	1	14%
	7.5 - 9 kips per square inch (50-60 megapascals)	3	43%
	9 - 10.5 kips per square inch (60-70 megapascals)	2	29%
	Other, please specify	1	14%
	Total	7	100%
INTL	confidential		

Prestressing

77. Are the sleepers pretensioned or post-tensioned?		
Pretensioned	8	100%
Post-tensioned	0	0%
Total	8	100%

78. What form of steel is used in the typical sleeper?			
	Wires	4	50%
	Strands	1	13%
	Bars	1	13%
	Other, please specify	2	25%
	Total	8	100%
INTL	confidential		
US	indented strand		

79. How many wires, strands, or bars pass through the centerline section of your concrete sleepers?	
INTL	confidential
	8
	20
US	20
	8
	18

80. What is the diameter of the wires, strands, or bars used?	
INTL	confidential
	0.30 inches (7.5 millimeters)
	0.11 inches (2.9 millimeters)
US	0.2094 inches (5.3 millimeters)
	3.0 - 8.0 inches (76.2 - 203.2 millimeters)
	5.32 inches (135.1 millimeters)

81. What is the jacking force introduced in the wires, strands, or bars?	
INTL	confidential
	12.6 kips (56.0 kilonewtons); wires: 211.8 kips per square inch (1460 newtons per square millimeter)
	80% of f_{pu}
US	7.0 kips (31.1 kilonewtons)
	100.1 kips (445.3 kilonewtons)
	6.8 kips (30.2 kilonewtons)

82. What is the yield strength of the wires, strands, or bars?	
INTL	confidential
	247 kips per square inch (1700 megapascals)
	270 kips per square inch (1862 megapascals)
US	265 kips per square inch (1827 megapascals)
	270 kips per square inch (1862 megapascals)
	260 kips per square inch (1793 megapascals)

Production

83. How are the concrete sleepers manufactured?		
Carousel	2	29%
Long line	5	71%
Other, please specify	0	0%
Total	7	100%

84. Is your typical sleeper manufactured to incorporate a specific fastening system?		
Yes	6	86%
No	1	14%
Total	7	100%

85. If so, what is that fastening system?	
INTL	Vossloh W14, Pandrol is also possible
	JR Central, JR Standard (drawings in spec)
US	any
	Pandrol Safelok III
	Fast clip / E clip
	Vossloh and Safelok III

86. How many sleepers did you produce last year?	
INTL	> 2 million
	180,000
	60,000
US	> 1 million
	15,000

87. What is your average daily production rate over the last five years?	
INTL	1200 in 3 shifts, 800 in 2 shifts
	200 (pretensioned)
US	3000
	50,000

Sleepers

88. Which infrastructure owners use your concrete sleepers?

INTL	see our reference list
	ÖBB, Wiener Linien, several private companies
	JR East, JR West, JR Central, Hokido North, South Kyushu, JR Shikoku
US	Public and private
	uprr
	CSX - LIRR
	BNSF UPRR

89. What is the design life of your concrete sleepers? (years)

INTL	it's more important what is the REAL life of the concrete sleeper
	50
	30 (often last 40)
US	50+
	25
	NA

90. Please provide design loads for your concrete sleeper.

	Question 90: What is the design axle load?	Question 90: What are the maximum design bending moments?	Question 90: What is the shear design load?
INTL	various	confidential	confidential
	27.6 tons (25.0 tonnes)	177.0 inch-kips (20.0 kilonewton-meters)	no issue
	27.6 tons (25.0 tonnes)	-	-
US	35.8 tons (32.4 tonnes)	381.0 inch-kips (43.0 kilonewton-meters)	-
	39.0 tons (35.4 tonnes)	Varies	-

Effectiveness

91. Have your sleepers ever experienced the type of deterioration as shown in the images below?

Yes	4	80%
No	1	20%
Total	5	100%

92. If so, what term would you use to identify this deterioration?

INTL	rail seat abrasion
US	rsa / rsd
	RSD
	Cavitation, Degradation

93. Please briefly describe the characteristics of this deterioration, in terms of where it occurred, at what rate it occurred, to what depth it occurred, etc.

INTL	Generally, rail seat abrasion is not a big issue in the EU. The abrasion on the pictures is not typical for us and we guess the reason are hard/stiff rail pads. The Austrian rail road company ÖBB is only using soft pads. Rail seat abrasion by rail is possible, but at first the pad be have been destroyed.
	Most of track is electrified (and signalled); stray currents jumping, affecting concrete, wires, and fastening
US	Elevated curves,deep south, unmaintained track, up to 1 inch (25.4 millimeters)

94. Please rank the following concrete sleeper and fastening system problems from most to least critical.

Top number is the count of respondents selecting the option. Bottom % is percent of the total respondents selecting the option.

	1	2	3	4	5	6	7	8
Derailment damage	0 0%	1 33%	1 33%	0 0%	1 33%	0 0%	0 0%	0 0%
Cracking from center binding	0 0%	0 0%	0 0%	0 0%	1 33%	1 33%	0 0%	1 33%
Cracking from dynamic loads	0 0%	0 0%	0 0%	0 0%	1 33%	1 33%	1 33%	0 0%
Cracking from environmental or chemical degradation	0 0%	0 0%	0 0%	1 33%	0 0%	0 0%	2 67%	0 0%
Deterioration of concrete material beneath the rail	2 50%	1 25%	0 0%	1 25%	0 0%	0 0%	0 0%	0 0%
Shoulder/fastening system w ear or fatigue	1 25%	2 50%	1 25%	0 0%	0 0%	0 0%	0 0%	0 0%
Tamping damage	2 50%	0 0%	1 25%	0 0%	0 0%	1 25%	0 0%	0 0%
Other (e.g. manufactured defect)	0 0%	0 0%	0 0%	1 33%	0 0%	0 0%	0 0%	2 67%

95. Of the following potential failure causes, please select any and all that have resulted in deficiencies of your concrete sleepers and fastening systems.

Deficient concrete strength	0	0%
Improper prestress force	0	0%
Poor material quality or behavior (of clamp, insulator, rail pad, or sleeper)	0	0%
Poor environmental conditions (e.g. moisture or fines intrusion)	1	25%
Manufacturing flaw s	0	0%
Improper component design (of clamp, insulator, rail pad, or sleeper)	0	0%
Fastening system damage	3	75%
Concrete deterioration beneath the rail	4	100%
Poor bonding of concrete to prestress	2	50%
Other, please specify	0	0%

190. Please enter the following general information. Any information obtained on this page will remain confidential and will not be released.

7 Responses

191. Please briefly describe the technical responsibilities related to your position.

6 Responses

192. If you are aw are of any other individuals w ho w ould be able to offer relevant information, please provide their names and e-mail addresses.

2 Responses

193. What proprietary restrictions exist w ith the information you have provided in this survey?

4 Responses