A quantitative evaluation of the impact of soft subgrades on railway track structure

by

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Abstract

The railway networks in Canada traverse from coast to coast, and pass over diverse terrain with large stretches of very soft soils, including clay deposits and peat formations. The majority of these lines were constructed approximately 100 years ago. Railway loads have since increased significantly, resulting in the imposing of higher loads on older infrastructure, particularly those constructed on soft soil foundations. This combination has resulted in the need to upgrade these lines to handle the increased loads and the expected increase of volume of traffic. The main challenge in upgrading these lines is the limited knowledge about the location of poor subgrades as the extent and relative stiffness of the foundation have not been mapped and documented over long distances. The lack of this information has also limited the studying of the influence of subgrade on track performance as well as quantifying the value obtained from the investment in improving track and substructure.

An extensive trial with a newly developed rolling deflection technology, over 12,000 km of track, was conducted to assess its potential to map the variability in subgrade conditions over long distances. It was evident from the collected data that unprocessed deflection measurements are heavily affected by the track surface condition such as joints and geometry irregularities so as to obscure the deflections because of poor subgrade support. A methodology was developed to minimize the influence of the surface condition that occur at short wavelengths and show the variations in track deflections because of changes in subgrade conditions which occur at longer wavelengths. The comparison of the processed data at different subgrade and geology condition confirmed that they are consistent with field conditions and are representative of the subgrade conditions.

Mapping the subgrade condition over extensive lengths of track presented the opportunity to investigate the impact of subgrade stiffness on the prevalence of track geometry defects and degradation of track quality indices (TQI). This investigation was consisted of the analysis of 800 km of subgrade data and track geometry measurement from two subdivisions from different physiographic region of Canada. The analysis showed that the geometry defects have a strong correlation with both subgrade condition and its variability whereas the TQI are only related to the variability of subgrade condition. These results showed that the locations that have a large deflection and a high variability in deflection are those that are difficult to maintain, and at which maintenance is not always able to keep up with the degradation of the track geometry. It also suggested the processed data from rolling deflection measurement systems provides an evaluation of the underlying causes that result in the degradation of track conditions and allow for the identification of sections where it most likely that maintenance will not always be able to keep up with degradation; even if maintenance has done so recently.

A methodology was also developed for quantifying the effectiveness of different methods used to improve the railway track performance on soft subgrades. This methodology is comprised of quantifying the changes in track stiffness from before and after vertical track deflection (VTD) measurements, and the evaluation of the roughness of the track that has developed since the track upgrades. A project was discussed as a case study to explain the steps of this methodology. The result showed that replacement of joints with heavier continuously welded rail (CWR) can reduce the track deflection up to 60%. The results of replacing the suggested 600 mm of subballast with 300 mm of subballast and a geogrid showed no change in the performance of the track under the CWR.

Preface

This dissertation is presented in the "paper-format" style. Chapter 3 is published online on July 2016, in the American Society of Civil Engineering (ASCE) Journal of Transportation Engineering as: Roghani, A. and Hendry, M.T. "Continuous Vertical Deflection Measurements to Map Subgrade Condition along a Railway Line: Methodology and Case studies". Chapter 4 is accepted (on November 10, 2016) for publication in the American Society of Civil Engineering (ASCE) Journal of Transportation Engineering as: Roghani, A. and Hendry, M.T. "Quantifying the impact of subgrade stiffness on track quality and the development of geometry defects". I was responsible for all work reported in the manuscript including the data collection, data analysis and interpretation, discussion of the results, and manuscript composition. Dr. M.T. Hendry was involved in the concept formation and manuscript composition and as the supervisor, has reviewed all parts of the work.

To my parents

Acknowledgment

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1. Chapter 1: Introduction

Canada has one of the most extensive rail networks in the world, with 48,000 route kilometers of track. The railway network is well positioned to take full advantage of both the continental and international economy. The Canadian railway network is primarily a heavy freight railway network, which is typical for North America. However, it is subject to more severe climatic conditions than found elsewhere in North America.

Canadian railways traverse from coast to coast and pass over a wide variety of terrain. The prevalence of large stretches of glacio-lacustrine clays and very soft peat/muskeg subgrades is of particular concern. The subgrade forms the foundation of a railway network, and the performance of the subgrade affects the performance of all other components of the track structure. Soft subgrades have been linked to increased wear and degradation of track and ballast, resulting from large movements associated with a low track modulus (Hendry et al., 2008, 2011; Selig and Waters, 1994; TSB, 1999). Additionally, these soft subgrade materials are prone to ongoing settlement and plastic deformation, which can lead to sudden failure and safety issues for rail operations. The failure of embankment constructed on soft subgrades has been identified as the root cause of several incidents, such as derailment of trains at the Levis subdivision in southern Quebec in 1999 and 2004 (Konrad et al., 2007; TSB, 2008). In addition to the safety concerns, the cost of operation is usually higher for those lines that are constructed on soft subgrade as they require more frequent maintenance due to higher rate of geometry degradation. Even in extreme cases, the maintenance is not always able to keep up with the degradation of track geometry. The major challenge in rehabilitation of these lines is that the location of soft subgrades has not been comprehensively mapped and, consequently, their impact on track performance and the associated risk is not known to railway personnel.

This research program is the continuation of a study at the University of Alberta initiated following recommendations by the Transportation Safety Board (TSB, 2008) that state *"[t]he railway* *inspection technologies and procedures, mainly based on evaluations of track conditions at surface level, were not effective to assess the condition and the behaviour of the subgrade and detect the impending risk of collapse".*

The overall goals of this research program are to use a newly developed and advanced rolling deflection measurement technology to develop a methodology to map the subgrade condition over long distances; to quantify the significance and influence of subgrade quality on track outages, including the generation of geometry defects and degradation of track quality indices; and to quantify the effectiveness of remediation methods that are commonly used by the railway industry to enhance the performance of track constructed on soft subgrades.

1.1. Description of problem

The majority of the rail network in North America was constructed more than 100 years ago when route selection was mostly based on the way that required the least amount of earth work and the minimum number of bridges and tunnels; not much attention was given to the type of terrain or soil that the track was traversing (Li et al., 2015). As a result, large stretches of track are constructed over soft subgrade, including glacio-lacustrine clays and very soft peat/muskeg soils. With axle loads and traffic volume continuing to increase, the safety of rail tracks constructed over these soft subgrades has become a major concern for the railway industry in Canada and has resulted in the need to rehabilitate these sections of track. The challenge in the rehabilitation of these sections is the limited knowledge about the location of soft soils and the composition of the embankment. There is also limited knowledge of the history of track performance on the soft soils as it has not been rigorously documented.

The extent and relative stiffness of rail track foundations have not been comprehensively mapped due to the lack of an economical method to measure the track stiffness (the combined stiffness of the track and soft foundation) over long distances. New technologies have been developed in Sweden (Berggren et al., 2005) and at the University of Nebraska (MRail) (McVey et al., 2005;

Lu, 2008; Greisen, 2010) that may allow for the measurement of track vertical deflection under heavy axle loads. In addition, continuous readings from across the network from track geometry cars have the potential to provide a quantitative evaluation of track performance at *all* problematic sites, allowing for correlations between track quality and subgrade conditions.

During the course of this research, an extensive trial of the MRail system was conducted over more than 12,000 km of track from 14 different subdivisions in Canada to evaluate its potential for mapping soft subgrade location. This large database was used to develop and validate a methodology that quantifies the subgrade/substructure quality. Quantification of the subgrade information over extensive lengths of track has presented the opportunity to investigate the impact of soft subgrades on the performance of track with a focus on development of geometry defects and degradation of track quality indices.

1.2. Research objectives

The overall objectives of this research program are to provide a greater understanding of the significance and influence of rail track's substructure on railway operations and to develop a framework that can be used by industry to quantify the substructure conditions so that they can take measures to increase the safety and the reliability of the railway network.

The specific research objectives of this PhD program are as follows:

I. To develop the means to map the extent and stiffness of soft subgrades beneath rail structures so as to determine the extent and relative quality of these structures.

II. To examine the historical performance of the existing rail over various subgrade conditions with a focus on quantifying the effect of subgrade stiffness on generation of geometry defects and degradation of track quality indices.

III. To quantify the effectiveness of different remedial methods used to upgrade the performance of rail tracks at soft subgrade sections, and to discern the effect of the components of the substructure on the performance of the whole of the track structure.

1.3. Description of study sites

The data included in this research come from four railway lines in Canada: two mainline tracks and two branch lines. Canadian National's Lac La Biche subdivision (LLBS) and Canadian Pacific's Pierre to Rapid City subdivision (PRCS) are the two branch lines of this study. These two subdivisions are examples of branch lines on which significant increases of traffic and axle load are proposed. The extensive length of soft subgrade over these two subdivisions provided an opportunity to evaluate the potential and limitations of MRail measurements and to develop and validate a methodology to map the subgrade condition over long distances (corresponding to Objective I).

The two mainline tracks of this study, with a total length of 800 km, are two high traffic and heavy axle load subdivisions that are separated by several thousand kilometers and located in different physiographic regions of Canada with different subgrade types. The first subdivision is located within the interior plains (Prairies) and the second subdivision within the Canadian Shield. Three years of track geometry measurements from these two lines were used to quantify the impact of subgrade condition on generation of geometry defects and poor track quality (corresponding to Objective II).

A rehabilitation project was carried out over a 600 m section of track along the PRCS where several different remediation methods were employed. The vertical track deflection and track geometry measurement recorded before and after the rehabilitation were used to quantify the effectiveness of the upgrade methods (corresponding to Objective III).

The following is a brief description of the branch lines used in this study.

1.3.1. Lac La Biche subdivision (LLBS)

The LLBS is 438 km (274 miles) long, was constructed between 1914 and 1919, and runs from Edmonton to Fort McMurray, Alberta (**Figure 1-1**a). CN reacquired the LLBS from a short-haul operator (Athabasca Northern Railway and Lakeland & Waterways Railway) in 2008. The terrain through which the LLBS is constructed has low to moderate relief, with extensive flat-lying (i.e., poorly draining) areas. This terrain and the colder climate result in expansive muskeg (swamp) areas, and thus the track has long sections with very soft organic subgrades (Roghani et al., 2015). Up to 120 km of the 396 km railway line is estimated to be constructed over very soft muskeg foundations (**Figure 1-1**b) (Hendry et al., 2013).

These soft subgrades have made the LLBS challenging to maintain over time. The line was set to be abandoned by its previous owners in 2007 due to the requirement for extensive rehabilitation. Large sections of the railway line ran over thin layers of sand and gravel without a proper railway ballast material. Since reacquiring the line, CN has significantly invested in improvements to the structures to allow heavy trains to safely average 40 km/h between Fort McMurray and Edmonton (Bourgonje and Scott, 2011).

The LLBS is presently limited to carrying 121.5 tonne (268 kips) carloads and trains lengths of 90 cars; typically, one to two trains run per day. CN is considering plans to increase the allowable carloads to 130 tonnes (286 kips), lengthen trains, and run up to ten trains per day to meet the needs of the growing resource industry in this region. The goal of this investment is "*the pipeline on rails*" concept, which would allow trains consisting of 100 or more tank cars to deliver oil from oil production facilities in Fort McMurray to the Gulf of Mexico refineries in eight to ten days (Alberta Oil, 2009).

In October 2013, an innovative and large-scale investigation was initiated by CN to assess the current condition of the line and to estimate the performance under the proposed higher axle loads. The details of the extent of the investigation, the different methods of measurements, and the challenges encountered in the application of these measurements for assessment of the track can be found in Appendix A (Roghani et al., 2015).

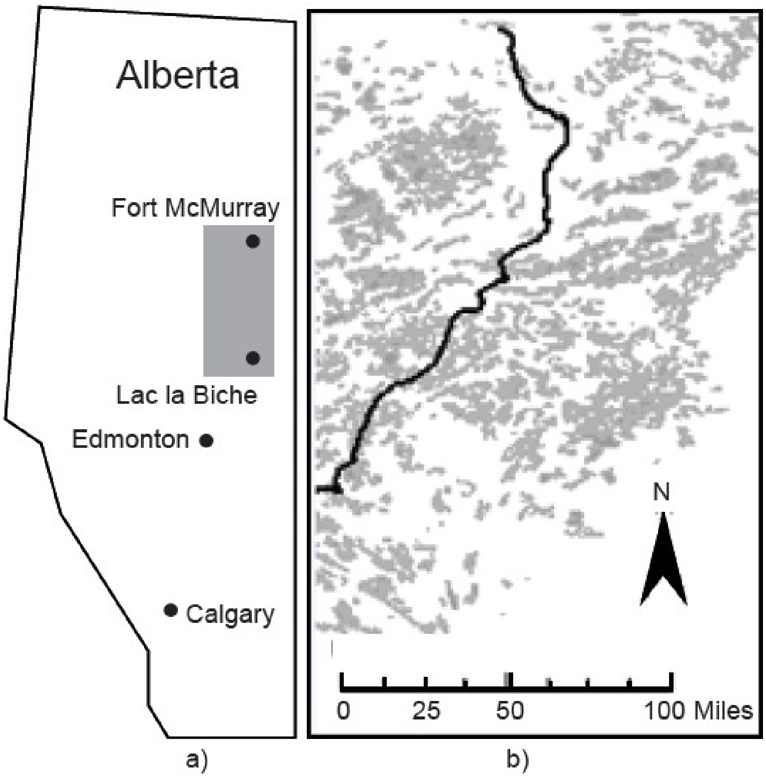

Figure 1-1. (a) Location of the LLBS within Alberta and (b) muskeg terrain (peatland) along the LLBS (*after* Roghani et al., 2015)

1.3.2. Pierre to Rapid City subdivision (PRCS)

The PRCS is 274 km of track connecting the cities of Pierre and Rapid City in South Dakota. This subdivision is operated by the Dakota, Minnesota and Eastern Railroad (DM&E). When the measurements were taken for this research, the DM&E was a subsidiary of Canadian Pacific (CP), which acquired the DM&E in 2007/08 with the intent to extend the railroad into the Powder River Basin and the nearby large coal mines. CP started extensive track bed upgrading, with the rehabilitation project conducted over 600 m section of track between MP 503.6 and 504 in the summer of 2012. The DM&E was purchased by short line operator Genesee and Wyoming Inc. in 2014.

The terrain that the PRCS passes through is characterized by the presence of shales of the Pierre formation (known as Pierre shale) (Searight 1952) (**Figure 1-2**). The Pierre shale is a thick shale formation (up to 600 m thick) of Upper Cretaceous age. It is characterized as a formation of plastic clays containing calcareous concretions and zones of chalk (Searight, 1952). Weathering and moisture turns this shale into soft, high plastic clay. This challenging subgrade has limited the performance of track along the PRCS and, as a result, the PRCS is classified as ‘excepted track’ (the lowest FRA classification), carries a 16 km/h (10 mph) speed limit for freight trains, and cannot be used by revenue passenger trains.

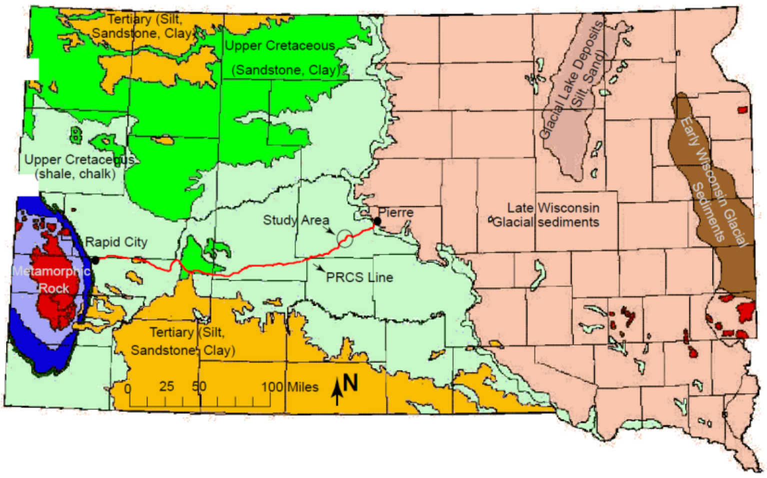

Figure 1-2. Location of the study area, and the surface geology of South Dakota (*after* South Dakota Department of Environment and Natural Resources 2013; Searight 1952).

1.4. Scope and methodology

This research aims to develop a methodology to map the location of soft subgrade/track over long distances, quantify the impact of soft subgrade on rail track structure by finding trends and correlations in the available field data, and develop a framework for quantifying the effectiveness of different remediation methods employed by the rail industry to improve the track stiffness. Two types of data will be considered in this study: vertical track deflection (VTD) measurements from the MRail system recorded under heavy axle loads and track geometry measurement data regularly collected by rail companies to monitor the performance of the track structure. In addition, the data from those methods that are used to determine the composition of the embankment, such as aerial photographs, surface geological mapping, the Ground Penetrating Radar (GPR), and shallow sampling, will be also included. The following steps were carried out to achieve the research objectives.

1.4.1. Mapping of subgrade stiffness on a large scale

This section describes the methodology followed for the completion of Objective I of this research, namely using the VTD measurements from the MRail system to map the stiffness of the subgrade beneath the rail tracks such that susceptibility maps can be produced to quantify the exposure of the railway network to soft subgrade issues. The MRail system has several advantages that make it an effective system for the purpose of this study: 1) this system provides continuous measurement of the entire subdivision and makes large-scale comparisons possible; 2) the system can be placed within a revenue service train and run at normal track speed and, thus, no track down time is required (which makes it economical for the railway industry); and 3) the measurements are taken under heavy axle loads so the impact of subgrade is implied within the measurements.

The scope of the work consisted of analyzing the data resulting from testing the system over the two subdivisions known for long sections of very poor subgrade: Lac La Biche and Pierre to South Dakota. The two study sites selected are examples of branch lines on which significant increases of traffic and axle load are proposed. The investigation includes: 1) analyzing the magnitude and wavelengths of the datasets to examine the extent of the impact from track surface components; 2) simulating the MRail rolling deflection measurements system to compare the extent of the impact from substructure components of the track; 3) employing a filtering method to eliminate/mitigate the impact of components other than subgrade from the measurements; and 4) comparing the processed data with the results of GPR surveys, surface geological maps, and shallow samples to validate the location of poor subgrades. The result of this section is also presented in map format to show the location of soft foundation along a subdivision. These maps provide the industry with the locations of sections of track that are most susceptible to track issues.

1.4.2. Impact of soft subgrade on track performance

This section describes the methodology followed for the completion of Objective II of this research, namely to quantify and provide greater understanding of the impact of soft subgrade on the performance of the rail tracks. The focus of this section is to examine the historical performance of the existing rail over various subgrade conditions to understand the long-term impact of the soft foundations on the rail tracks. The performance of the track included assessments that considered the track condition in terms of two parameters: track quality indices, which represent the track roughness (unevenness), and track geometry defects. These parameters are critical for the rail industry as they are indicative of track performance and safety. Track quality indices are the key parameters influencing maintenance requirements, the rail-track interaction, and impacts the magnitude of stress on the foundation. In addition, the rate of the development of track geometry defects in the track is important as track geometry defects are the second leading cause of derailments in both the United States of America and Canada (Liu et al., 2012; TSB, 2013).

The scope of the work includes combining the result of mapping the subgrade stiffness with three years of geometry measurements from two mainline subdivisions to quantify how the soft subgrade has impacted the track performance.

1.4.3. Quantifying the effectiveness of upgrade methods

This section describes the methodology followed for the completion of Objective III of this research, namely to evaluate the effectiveness of remedial methods used to upgrade performance of track constructed on soft subgrades. The main focus of this section is to compare the performance of several sections of the PRCS before and after rehabilitation. Rehabilitation consisted of reconstruction of the track embankment with layers of new ballast and subballast, installation of geogrid at the subballast and subgrade interface, and using heavier welded rails. To accomplish this, the VTD and track geometry measurements recorded before and after rehabilitation were compared to quantify the extent of the impact of each upgrade method.

The scope of the work includes: 1) comparing the result of track quality indices after remediation to quantify the rate of track degradation for each test section; and 2) comparing the result of VTD measurements before and after remediation to quantify the percent improvement in track deflection and effectiveness of the methods in stiffening the track.

1.5. Overview of thesis

This thesis has been prepared in a paper-based format. The thesis consists of six chapters, including this first introductory chapter, and two appendixes:

Chapter two presents the necessary literature review for this study.

Chapter three (manuscript #1) presents the methodology developed for the use of continuous vertical track deflection measurements from a moving loaded rail car to map the subgrade condition along a railway line. This analysis of deflection data was developed following the collection of over 12,000 km of measurements. A filtering method is proposed to eliminate the impact of surface conditions on the VTD measurements. The resulting processed data are compared with the geology over two study railway subdivisions to demonstrate that the processed vertical track deflection measurements are representative of the subgrade conditions. The filtering process, examples of the impact of track surface on the deflection measurements, and the limitations of the resulting data are also discussed in this chapter.

Chapter four (manuscript #2) quantifies the impact of subgrade condition on track performance. The data included in this chapter consist of 800 km of VTD and track geometry measurements. The VTD measurements are used to define two indices that represent the subgrade stiffness and its variation over the railway lines. The subgrade information is then compared with the location of geometry defects and variation of track quality indices. The detailed quantification of the extent of impact of subgrade stiffness on geometry defects and track quality indices are presented in this chapter. A different approach on combining the track geometry and track deflection measurement is presented in Appendix B along with an example on how to implement these information into maintenance planning.

Chapter five (manuscript #3) a methodology is developed for quantifying the effectiveness of different methods used to improve the railway track performance on soft subgrades. This methodology is comprised of quantifying the changes in track stiffness from before and after VTD measurements, and the evaluation of the roughness of the track that has developed since the track upgrades. A project is also discussed as a case study to explain the steps of this methodology.

Chapter six provides conclusions and recommendations.

Appendix A presents the extent of the investigation over the LLBS conducted by CN in October 2013 to assess its current condition and estimate the performance under the proposed higher axle loads. The details of different measuring systems used in the investigation and the challenges encountered in the application of the measurements for assessment of the track structure are also discussed.

Appendix B presents a comparison between the VTD and track geometry measurements over the LLBS. The results of this comparison are used to quantify the relationship between track stiffness and roughness and the occurrence of track defects over a branch line. This relationship is further used to define threshold values of track roughness and stiffness, and propose a hazard chart for maintenance requirements along the Lac La Biche subdivision.

Note: *As the track specifications and standards in the North America are based on imperial units, in this thesis and for the convenience of the readers, all the values are given in both metric and imperial units.*

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2. Chapter 2: Literature Review

2.1. Track structure

The main function of a rail track is to provide a durable, smooth running surface for the trains and to distribute the wheel loads to a sufficiently low pressure on the subgrade, the weakest component (Kerr, 2003). A typical construction of a railway track consists of superstructure and substructure (Figure 2-1) (Selig and Waters (1994)). The superstructure is defined as the rails, fastening system, and ties and the substructure consists of the ballast, subballast, and subgrade. As many of the issues associated with the substructure are geotechnical in nature and are affected by the presence of water, drainage is often considered a component of the substructure (Berggren, 2009). The following sections describe the main functions of substructure components of a railway track.

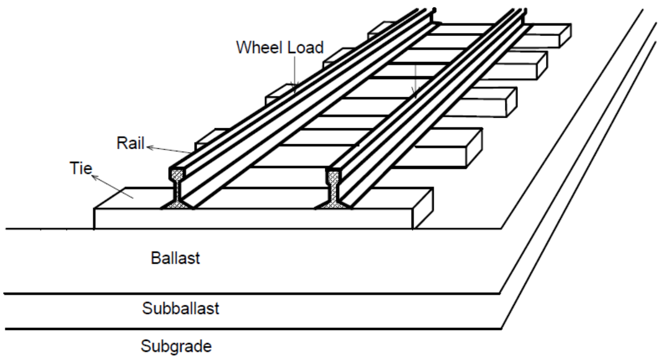

Figure 2-1. Typical construction of a railway track structure showing both the superstructure and substructure (*after* Selig and Waters, 1994)

2.1.1. Ballast and subballast

The ballast is crushed granular rock material beneath and surrounding the ties that lies at the interface of the track structure and underlying foundation. Ideally, ballast has a uniform grading; the industry standard is an average particle size of 35 mm. The ballast has multiple functions, including (Selig and Waters, 1994):

i. Maintaining the track geometry by resisting the lateral and longitudinal forces generated by the moving trains.
ii. Spreading the load of the train over the subgrade to prevent overstressing.
iii. Allowing for quick drainage of water from the embankment structure.

The contamination of ballast voids by fine particles (fouled ballast) greatly affects ballast functionality in the field. The main sources of fine particles among ballast aggregates are (Kassen et al., 1987):

i. Aggregate breakage due to repeated traffic loading, in particular heavy axle loads.
ii. Migration of fines from the subgrade (mud pumping)
iii. Weathering, including the effect of freeze-thaw and thermal effects.
iv. Maintenance activities including tamping.

According to Selig and Waters (1994), the major portion of fines (>76%) is from breakage and deterioration of ballast aggregates as a result of passing heavy freight loads and high speed trains. Therefore, railroad companies search for quarries that can economically provide a geotechnically suitable ballast material. Mechanical abrasion tests are commonly used to evaluate the suitability of rock for use as railroad ballast. There are three main types of abrasion tests on ballast: Los Angeles Abrasion (LAA), Micro Deval, and Mill Abrasion. The American Railway Engineering and Maintenance of Way Association (AREMA) manual states that LAA is the best way to measure the potential breakdown of ballast materials under loading.

The subballast layer is a transition material between the ballast and the subgrade; ideally, it is a well-graded material that is designed to further distribute the load, drain freely, and prevent the migration of subgrade material into ballast.

2.1.2. Granular layer thickness

Ballast and subballast together form the granular layer below the tie and above the subgrade. This layer should be thick enough to lower the stress on top of the subgrade to a tolerable level so that it prevents the failure of subgrade by excessive deformation (plastic strain) or progressive shearing of the soil (Selig and Waters, 1994). The thickness of the granular layer also affects the track modulus and thus the track vertical deflection.

The thickness of the ballast depends on many factors, such as ballast quality and maximum axle load of the line. A minimum of 0.3 m (12 in) is generally considered sufficient as per industry experience (Raymond, 1978). AREMA (2012) recommends the use of Talbot's equation to calculate the minimum depth of granular material:

$$h = (16.8\ \frac{p_a}{p_c})^{0.8}\ , \qquad \text{Equation 2-1}$$

where h is the thickness of ballast and subballast below the cross tie (in), p_a is the stress at the bottom of the tie (top of ballast) (psi), and p_c is the maximum allowable stress on the subgrade (psi). p_a is calculated using

$$p_a = \frac{2w(1+IF)DF}{A}\ , \qquad \text{Equation 2-2}$$

where w is the wheel load, IF is the influence factor (function of speed and wheel diameter), DF is the distribution factor (function of tie spacing), and A is the surface area of the base of the tie. p_c is the most challenging parameter to determine in this equation. AREMA (2012) recommends that the stress on the top of the subgrade should not exceed 172 kPa (25 psi). When test results of field measurements are available, the following equation can be used:

$$p_c = \frac{CU \times N_c}{F.S.}\ , \qquad \text{Equation 2-3}$$

in which CU is the undrained shear strength, N_c is the bearing capacity factor, and $F.S.$ is a factor of safety (2-5 according to AREMA). The US Army Corps of Engineers (2003) suggests an $N_c$value of 2.8 for unreinforced sections and 5.8 when the aggregates are confined by a biaxial geogrid.

Another method for calculating the granular thickness is proposed by Li and Selig (1998a, 1998b). The method is based on the combined use of a multilayer linear elastic finite element model coupled with extensive laboratory testing. Their methodology is based on the intention to prevent progressive shear failure and excessive plastic deformation. In their approach, the ballast thickness is determined by limiting the plastic strain for a given number of cycles and the axle load to a certain value.

2.1.3. Subgrade

Subgrade forms the foundation of a railway track and its quality has a direct impact on the performance of all track components. Track geometry is hard to maintain if the subgrade does not have enough bearing capacity and, consequently, the track will need frequent maintenance. Soft subgrades have been linked to increased wear and degradation of track and ballast resulting from large movements associated with soft subgrades (Hendry et al., 2008, 2011; Selig and Waters, 1994; TSB, 1999). Additionally, soft subgrade materials are prone to ongoing settlement and plastic deformation, which can lead to sudden failure and safety issues for rail operations (Konrad et al., 2007; TSB, 2008). Despite its importance, the substructure (particularly the subgrade) has historically been given less attention than the superstructure (Selig and Waters, 1994).

The stiffness of subgrade under a railway track is commonly characterized by its resilient modulus (Li et al., 2015). The difference between Young's modulus and the resilient modulus is presented in Figure 2-2. The moisture content and fines content are the most important parameters affecting the resilient modulus. Generally, increases in moisture and fine content appear to decrease the resilient modulus (Doung et al., 2015; Thom and Brown, 1987; Kamal et al., 1993).

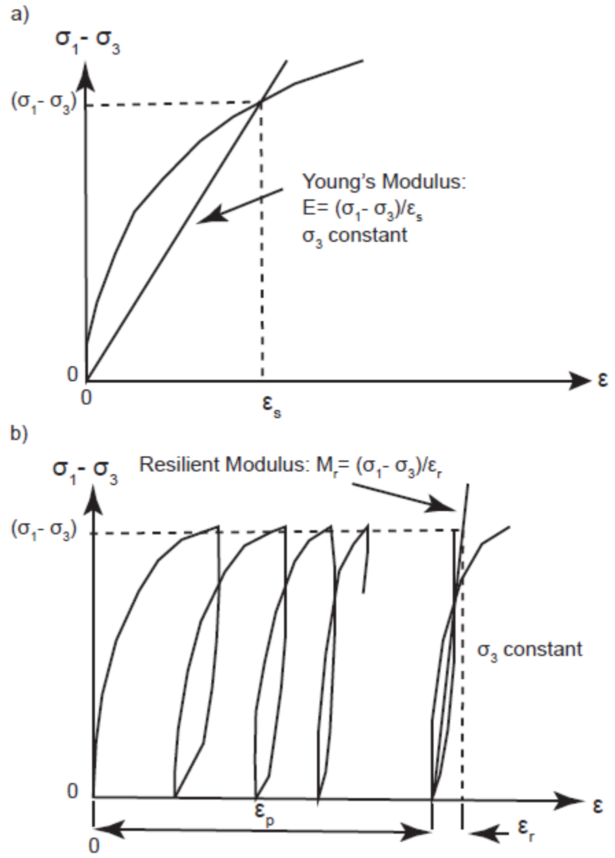

Figure 2-2. Definition of a) static Young modulus and b) resilient modulus (*after* Li, 1994)

Fine grained soils with high clay content are one of the problematic subgrade types for railway track. Progressive shear failure and excessive plastic deformation, which occur due to the repeated overstressing of the subgrade (Li et al., 2015), are two of the most common modes of failure of this type of soil (Figure 2-3). Improving soil strength characteristics by injection or reducing the deviator stress on top of the subgrade by increasing the granular layer are the main approaches taken to avoid these type of failures. Muskeg (peat) is another type of problematic soil for railway foundations, especially within the Canadian rail network. The common view in geotechnical engineering is that muskeg should be avoided as a foundation material whenever possible. Due to the vast expanse of muskeg-covered terrain in Canada, avoidance is not always an option, particularly for continuous linear structures such as railways (Hendry and Roghani, 2015). Muskeg undergoes large deformations when loaded. Cyclic loading by trains has been observed to result in displacements of the track in excess of 25 mm (1 in). In addition, the stability of structures constructed over this type of soil is very difficult to ascertain. Figure 2-4 shows excessive plastic deformation for a section of track constructed on top of muskeg. The pore water pressure and deflection response of muskeg under repeated loading from trains as well as the laboratory behavior of muskeg have been extensively studied (see Hendry et al., 2012, 2013, 2014, Acharya, 2016).

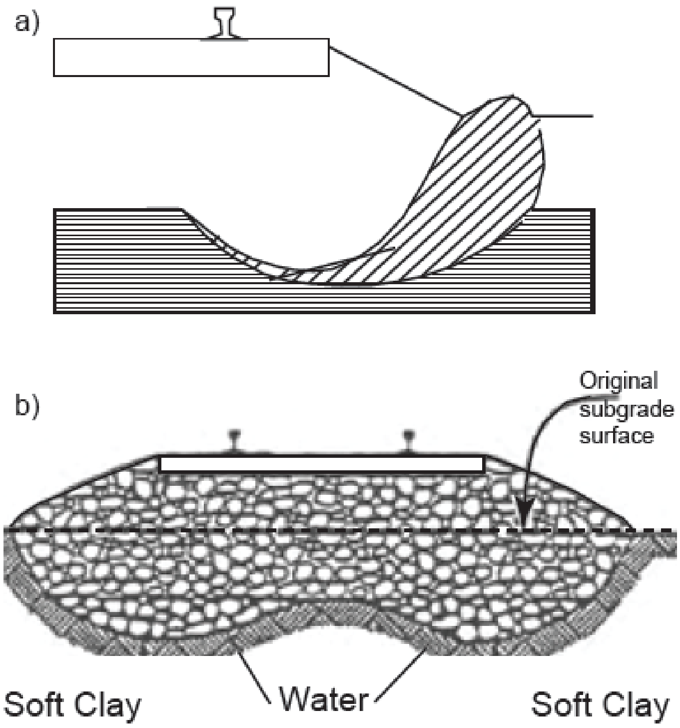

Figure 2-3. The two most common modes of subgrade failure under a railway track: (a) progressive shear failure and (b) excessive plastic deformation (*after* Li et al., 2015)

Figure 2-4. Excessive plastic deformation of a section of track constructed on muskeg

2.1.4. Drainage

There are three main sources of water that affect rail track: direct water (rain or snow), runoff water (surface water or snowmelt), and ground water (water flowing up into the track) (Li et al., 2015). The lack of water drainage from any of these sources along with the cyclic loading due to trains passing have a strong influence on rail track performance. Hyslip and McCarthy (2000) found that inadequate drainage is a critical factor that leads to substructure problems. Even a small increase in the water content of the subgrade can result in a significant reduction in the bearing capacity of the soil and, consequently, excessive plastic deformation and failure (Ferreira, 2011). The softening impact of water is more profound in moisture sensitive soils such as clay; even stiff clayey soil can significantly soften in the presence of water and repeated loading (Ghataora et al., 2006). Therefore, appropriate drainage that keeps the water out of the track could play an important role in subgrade behaviour.

The presence of fine particles within the ballast, arising from ballast breakage or migration of fines from the subgrade, will reduce the void ratio of the ballast and, consequently, its drainage capacity. The results of experimental studies conducted by Su et al. (2015) show that the permeability capacity of ballast decreases as the fouling ratio increases. They also found that the water head influences the permeability of ballast but, at high fouling ratios, all permeability values move toward a common value that is close to the permeability of the fouling materials regardless of water head. Thus, maintaining a clean layer of ballast is important for adequate drainage of railway tracks. Mud pumping is often an indication of inadequate drainage at the ballast layer (Figure 2-5). Another factor that hinders drainage is excessive plastic deformation of the subgrade. Ballast material placed over time at the center of the railway embankment pushes the subgrade downwards and to the sides, resulting in some heaving under the embankment shoulders. This leads to ballast material accumulations beneath the centerline of the track, known as “ballast pockets”. **Figure 2-6** shows an example of ballast pocket formation under a rail track.

Figure 2-5. An example of mud holes formed in a soft formation near Swan landing on CN's Edson subdivision

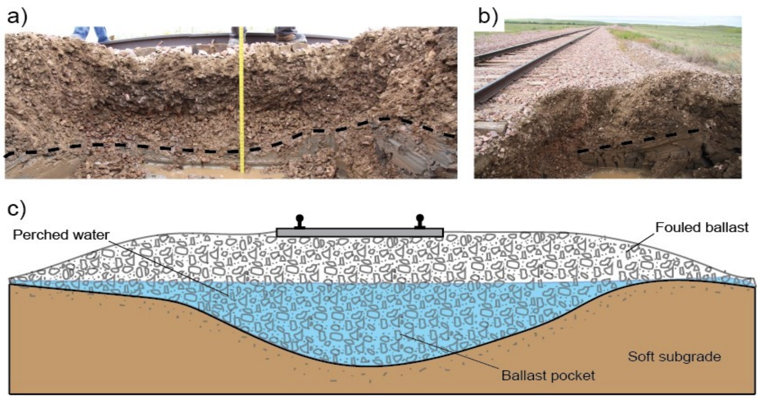

Figure 2-6. Railway embankment profile as observed in a test pit (a and b). The horizon between the ballast and subgrade is highlighted with dashed lines. (b) Sketch of the interpretation of the embankment section (not to scale).

2.2. Impact of track substructure on overall track performance

Track substructure issues manifest in the track surface as geometry defects and rough track geometry. The quality of track substructure affects the performance of the superstructure as the performance of the substructure and superstructure are interdependent; a decline in one component can limit the performance of others. For example, large deformations due to soft foundations will lead to wear and degradation of substructure and track elements (Hendry et al., 2008, 2011; Selig and Waters, 1994; TSB, 1999, 2008). Similarly, impacts due to rail surface imperfections can lead to the pumping of fine grained soils from the subgrade that contaminate the ballast; higher impacts can result in plastic deformation of weak subgrade soil. Thus, a greater understanding of track substructure condition is imperative for safe and reliable operation.

2.2.1. Track modulus as a measure of track substructure condition

Track substructure condition is usually quantified using the track modulus, which is a single parameter that reflects the effects of all track substructure components under the rail (Cai et al., 1994). The track modulus is the coefficient of proportionality between the rail deflection and the vertical contact pressure between the rail base and track foundation (Cai et al., 1994). In other words, the track modulus is the supporting force per unit length of rail per unit rail deflection (Selig & Li, 1994). The track modulus is a basic parameter of track design that influences the bearing capacity of track, the dynamic behaviour of passing vehicles, and, in particular, the quality of track geometry and the life of track components.

2.2.2. Theoretical calculation of track modulus

The bending theory of an elastic beam can be described by considering an infinite, continuous beam supported by an elastic foundation and subjected to a single point load. Figure 2-7 shows a free body diagram of a rail under a wheel load. The rail is considered to be a continuously supported beam, where x represents the distance along the bean and $w(x)$ represents the vertical beam deflection at point x. The structure underneath (including ties, fastener, ballast, and subgrade) supports the rail with a reaction distributed force, $p(x)$.

The difference in the vertical load ($q(x) - p(x)$) causes curvature in the beam, as given by the differential equation

$$\mathrm{EI}\,\frac{\mathrm{d^4w}}{\mathrm{dx^4}} + \mathrm{p(x)} = \mathrm{P}, \qquad \text{Equation 2-4}$$

in which E is the modulus of the elasticity of the rail, I is the moment of inertia, and x is the longitudinal distance along the rail. Most track models consider as their basic equation but differ in their assumptions about the reaction force ($p(x)$).

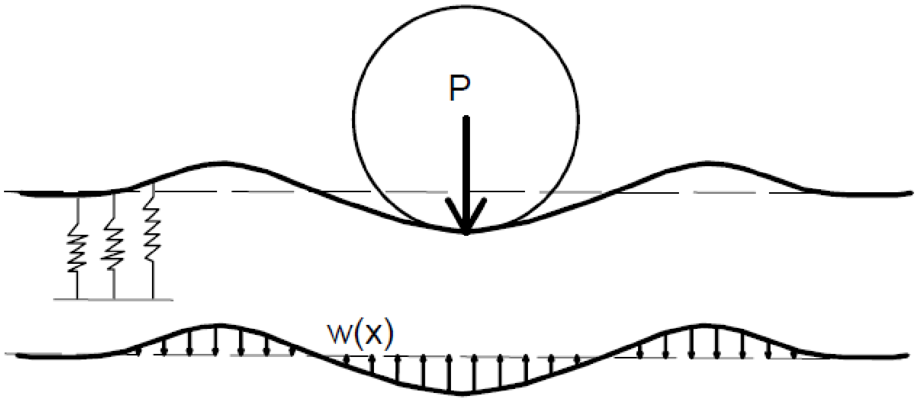

Figure 2-7. Free-body diagram of the rail (*after* Lu, 2008)

The beam on elastic foundation model, proposed by Winkler (1867), assumes that the supporting force of the track foundation is linearly proportional to the vertical rail deflection, ($p(x) = u\,w(x)$). Here, the coefficient u is defined as the track modulus. Substituting this assumption into Equation 2-4 and applying the boundary conditions of the beam and solving the differential equation yields

$$w(x) = -\frac{P\beta}{2u} e^{-\beta|x|}[\cos(\beta|x| + \sin(\beta|x|))]), \quad \text{Equation 2-5}$$

where $\beta = \sqrt[4]{\frac{u}{4EI}}$, P is the load on the track, u is the track modulus, E is the modulus of elasticity of the rail, I is the second moment of area of the rail section, and x is the longitudinal distance along the track.

2.2.3. Impact of track modulus on track performance

Both low track modulus and large variations in track modulus are undesirable (Read et al., 1994; Ebersohn et al., 1993). In general, a relatively high track modulus is beneficial as it provides sufficient track resistance to applied loads and results in decreased track deflection, which reduces track deterioration. However, too high of a modulus (too stiff) leads to fatigue, fracture, and excessive vibrations (Selig & Li, 1994). A low track modulus causes large deformations that subsequently increase maintenance (Read et al., 1994; Ebersohn et al., 1993). Large variations in track modulus, such as those often found near bridges and crossings, increase dynamic loading (Zarembski & Palese, 2003; Davis et al., 2003). The increased dynamic loading reduces the life of the track components, resulting in shorter maintenance cycles (Davis et al., 2003). Reducing variations in track modulus at grade (i.e., road) crossings leads to better track performance and less track maintenance (Zarembski & Palese, 2003). It has also been suggested that track with a high and consistent modulus will allow for higher train speeds and therefore increase both performance and revenue (Heelis et al., 1999).

2.2.4. Subgrade as the most significant parameter among substructure components

All track substructure components contribute to the track modulus and overall response of the track. Various parametric studies have been conducted to understand the significance of each substructure component on the track response (Stewart and Selig, 1982; Stewart, 1985; Selig and Li, 1994; Shahu et al., 1999; Shahin and Indraratna, 2006; Rose and Konduri, 2006). These numerical models are based on static analysis, and the influence of the dynamic effects are taken into account by the application of a dynamic amplification factor (DAF) that may range from 1.1 to 2.8 (Esveld, 2001; Priest et al., 2010). Some dynamic models have also been developed with a focus on the influence of dynamic loading on track structure (Krylov, 1995; Kaynia et al., 2000; Grabe, 2002; Yang et al., 2009). They suggest that resonance effects may cause large track displacements when the train speed approaches the Rayleigh wave velocity.

Despite the difference in their approaches, the main conclusion of the static analysis studies is that the subgrade has the greatest influence on the overall track response. Selig and Li (1994) show that an increase in subgrade modulus by a factor of 10 leads to an increase of track modulus by a factor of 8. They also show that, for a given subgrade modulus, the ballast thickness is the most significant variable influencing the track deflection under load (Figure 2-8).

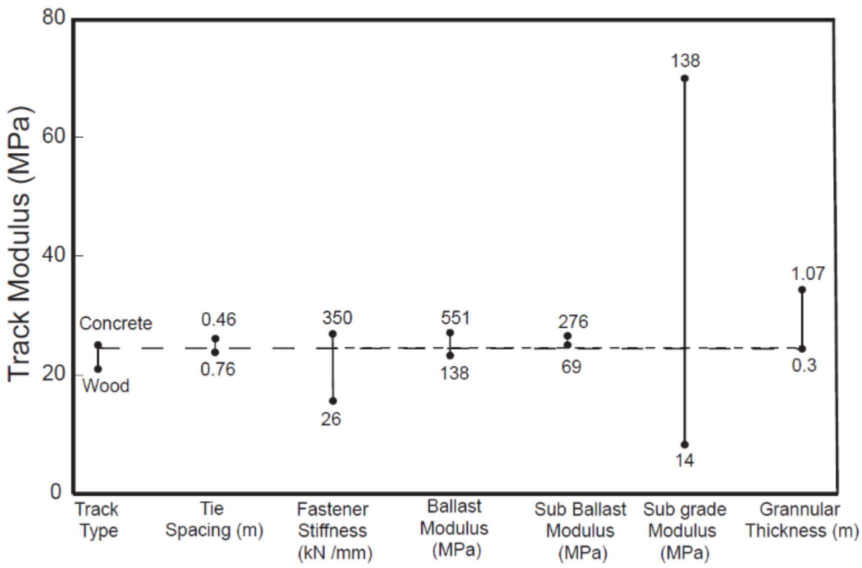

Figure 2-8. Impact of track components on track modulus (*after* Li and Selig, 1994)

The lesser influence of ballast and subballast compared to subgrade has been attributed to the fact that the depth of influence for ballast and subballast is much less than for subgrade (Li et al., 2015). The influence of subgrade condition is further enhanced by the fact that the subgrade modulus is the most variable, and least controllable, of all track components, whereas the ballast and subballast moduli do not vary much along the track. Therefore, a change of track modulus in the field is primarily considered as an indication of a change of subgrade condition (Selig & Li, 1994). Because the subgrade condition is subject to weather, extremes of temperature, and moisture, the track modulus may vary with seasonal changes. The soil parameters depend upon the physical state of the soil, the stress state, the soil type, and the moisture conditions (Li & Selig, 1995; Selig & Li, 1994).

2.3. Track geometry measurements

Track geometry measurements describe the position that each rail occupies in space and are used by the railway industry to ensure that the shape of the rail allows for the safe passage of trains at the designated maximum speed of the track (AREMA, 2012). The gauge, alignment, profile, crosslevel, and warp are the main geometry parameters used to define the shape of the rails, where the gauge is the distance between two rails measured 16 mm below the top of rail with standard gauge equal to 1435.1 mm; the alignment is the horizontal deviation of the gauge side (inside) of the rail from a line subtended from two points 18.9 m apart on this surface measured at the midpoint of that line (i.e., a mid-chord offset); the profile is the mid-chord offset measured vertically on the surface of the rail; the crosslevel is the elevation difference between both rails on a tangent track; and the warp (i.e., twist) is the difference in crosslevel values between two points located 18.9 m (62 ft) or 9.5 m (31 ft) apart along the track (AREMA, 2012; Federal Railroad Administration's Track Safety Standards Compliance Manual, 2007; Transport Canada's Rules Respecting Track Safety, 2011).

Track performance assessment is currently based in large part on the interpretation of track geometry measurements (Sadeghi and Askarinejad, 2010). This assessment is done by introducing two metrics: track geometry defects and the track quality index (TQI). The following sections describes the two metrics and how they are used to quantify track performance.

2.3.1. Track geometry defects

A track geometry defect exists when the measured values of track geometry exceed threshold values set within regulations (FRA, 2007; TC, 2011). These threshold values are defined based on an assigned class of track, where the class of track is defined so as to limit the speed of trains to match the condition of the track. According to both Transport Canada and the FRA, Class 1 has the lowest maximum track speed of 16 km/h (10 mph) and the highest geometry thresholds, and Class 5 has the highest maximum track speed of 129 km/h (80 mph) and the lowest geometry thresholds. A section of track is maintained to meet the requirements of its assigned class, and thus the maximum allowable speed. Those defects that exceed regulator limits are called urgent, as they are a safety issue and need an immediate action. In addition to safety limits, there is a second category with more stringent threshold values defined by the operator (typically 66 to 75% of the safety limits (Li et. al., 2015)) and represent the maintenance limit for the track. These are called priority defects and must be monitored until repaired to ensure they do not escalate and exceed safety limits and becoming urgent defects.

2.3.2. Track quality index (TQI)

TQI is another metric used to quantify track performance. TQI is calculated by aggregating the geometry measurements over segments of track. It summarizes the relatively large quantity of discrete measurements generated by a track geometry car to allow characterization of an entire track segment, thus facilitating comparison between track segments (AREMA 2014).

For high-level assessments of railway track performance or track 'quality', researchers and the industry have developed measures that aggregate the geometry of the track over segments of track (El-Sibaie & Zhang, 2004; FRA, 2005; Berawi et al., 2010; Sadeghi & Askarinejad, 2010). This aggregation has been done using simple statistical analyses (El-Sibaie & Zhang, 2004; FRA, 2004, 2005; Berawi et al., 2010; Sadeghi & Askarinejad, 2010), fractal analyses (Hyslip, 2002), and metrics of relative and running roughness (Ebersohn, 1995; Ebersohn & Conrad, 1998).

TQI is highly variable and changes over time. In addition, different track features (tangents, curves, switches, bridges, etc.) have different TQI ranges. Zhang et al. (2004) found that switches have the worst quality index, reflecting the discontinuities and difficulty of maintenance at these locations. On the other hand, bridges showed the best quality as a result of the high stiffness of these structures. For this reason, AREMA has special recommendations on track segmentation for calculating TQI and recommends that the type of track structure should be homogenous over the segment. In moving window methods of calculating TQI (such as running standard deviation), however, segmentation is not required and segments can include non-homogenous track structures.

TQIs have seen some application with respect to the scheduling of railway maintenance (Tolliver & Benson, 2010) and are typically used for (AREMA, 2014):

i. Prioritizing maintenance activities, such as tamping and undercutting;
ii. Re-classification of track or placing a slow order;
iii. Quantifying the rate of track degradation; and
iv. Assessing the effectiveness of maintenance or track renewal.

2.3.3. Track geometry measurements and track safety

Track geometry defects and TQI as two measures of track performance have significant impacts on track safety. Track geometry defects are an important measure of track safety as they are the second leading cause of derailments in both the United States of America and Canada (Liu et al., 2012; TSB, 2013). In the United States, 658 of the 1890 train accidents (34.8%) that have occurred since 2009 are attributed to track irregularities. These accidents resulted in $108.7 million in losses and damages (Peng, 2011). According to the Transportation Safety Board of Canada (www.tsb.gc.ca), about 14% of the main track incidents in Canada between 2004 and 2013 were related to track defects. Thus, detecting and repairing track geometry defects are critical for safe and effective railway operations and can effectively reduce the probability of derailments. TQI as a parameter that describes the roughness of track geometry is important for track safety. Rough track geometry increases the dynamic component of the wheel-rail contact force (He et al., 2014) and can lead to deterioration of track components and rolling stock and, in extreme cases, even derailments (FRA, 2000).

2.4. Vertical track deflection measurements

Several vertical track deflection (VTD) measurements systems are used by the railway industry. The main goal of these methods is to assess the potential of a railway line for increases in axle load or level of traffic. These methods are divided into those that take measurements at discrete locations and require the measurement system to stop to take measurements and those that allow for the measurement of the VTD while continuing to move along the track.

Standstill methods have been more widely used whereas rolling measurement techniques have been more recently developed and are still the subject of research (Berggren, 2009). In standstill methods, the stiffness of a section of track is calculated by instrumenting either the ties and/or rails with transducers or accelerometers and monitoring the response under a known load. A number of different techniques, such as the Falling Weight Deflectometer (FWD) and the impact hammer, have been developed based on static loads (Zarembski & Choros, 1980; Kerr, 1983; Ebersohn & Selig, 1994; Read et al., 1994) and are considered standstill devices for measuring track stiffness. The continuous measurement of track support over long distances has the potential to be a significant addition to the maintenance tools available to railroad personnel (Sussman et al., 2001; Ebersöhn et al., 1993; Carr, 1999). Researchers have focused on developing automated measurement technologies to evaluate track modulus and stiffness using moving loaded vehicles (Li et al., 2004; FRA, 2004; Norman et al., 2004) or vibrating masses (Wangqing et al., 1997; Berggren et al., 2002). This section presents a summary of the major measuring systems that are commonly used in the North American rail industry.

2.4.1. Falling weight deflectometer (FWD)

The FWD is commonly used to measure the stiffness of the track structure excluding the rails (Burrow et al., 2007). This is a modified version of the FWD systems previously developed for use on roads and highways. The assessment of the results of FWD testing is based on the assumption that structural integrity is inversely proportional to the amount of surface deflection observed under an applied impact load (Meier & Rix, 1993). The FWD consists of a mass that is dropped from a known height onto rubber buffers mounted on a footplate (Burrow et al., 2007). A load cell on the center of the plate measures the resulting impact and the velocity transducers are used to determine surface velocity at various distances (Figure 2-9) (Burrow et al., 2007). The velocities are integrated to give vertical displacements. This application of an impact load has been suggested to produce a load pulse similar to that applied by a single axle of a train travelling at high speed (Sharpe & Collop, 1998). The maximum deflection of the track at 300 and 1000 mm from the drop (d300 and d1000 as shown in Figure 2-9) gives an indicative value for the deflection of the granular layer (ballast and subballast) and the subgrade, respectively. The FWD system has been used for several decades in the United Kingdom (UK) to evaluate the quality of various track bed layers. **Figure 2-10** shows the vertical deflection measured by geophones at the locations shown in **Figure 2-9**, as measured along a section of track in the UK (Brough et al., 2003). Given that d_{1000} is indicative of the subgrade condition, tracking the changes in this parameter along a section of track shows the variation in subgrade stiffness. Measuring track modulus at single locations by static methods is a time-consuming process. In addition, static methods are not viable for investigating track conditions on a subdivision scale.

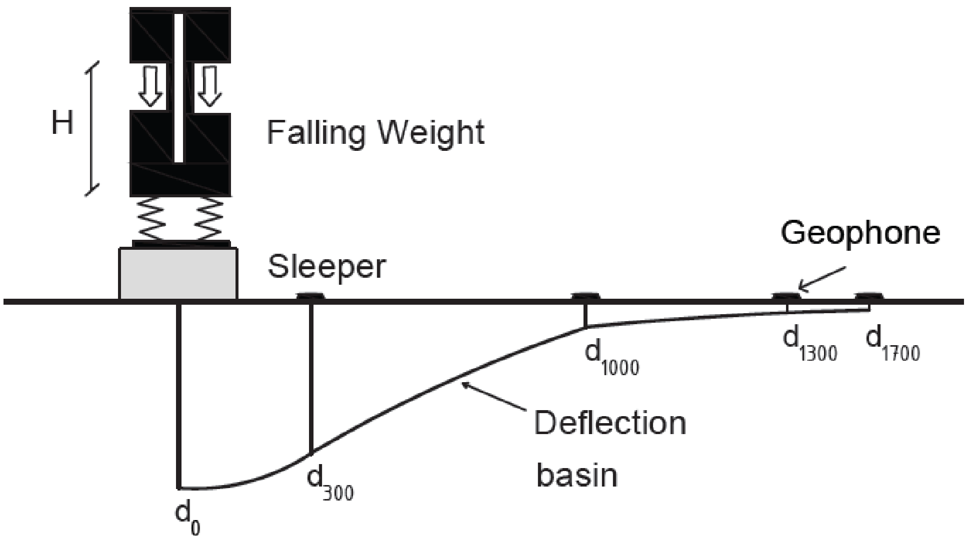

Figure 2-9. Simplified schematic of an FWD measurement system (*after* Burrow et al., 2007)

2.4.2. Track loading vehicle (TLV)

To improve track maintenance efficiency and railroad operational safety, the Transportation Technology Center Inc. (TTCI) developed the Track Loading Vehicle (TLV) to continuously measure track stiffness (Figure 2-11) (Thompson et al., 2001, Li et al., 2002, 2004). Their measurement concept contains one instrumented coach (the TLV) and an additional empty tanker car used to measure the unloaded vertical rail profile. The coach axle can carry a load of 4 to 245 kN (1 to 55 kip) and the tanker car (reference axle) applies loads of 0 to 13 kN (0 to 3 kip). Laser-based systems on each vehicle measure the track deflections associated with the applied loads. **Figure 2-12** shows the result of a TLV survey obtained over two separate runs of the same section with vertical loads of 44 and 178 kN (10 and 40 kips), respectively (Li et al., 2004). The results of contact deflection, shown in **Figure 2-12**b, were obtained by calculating the difference between the displacements measured under 178 vs. 44 kN loading The results are interpreted based on the hypothesis that the displacements that result from the 44 kN loading are primarily from the rail, tie, and ballast, while the displacements that results from the 178 kN load are from both the track structure and the subgrade (Li et al., 2004). **Figure 2-12**b is obtained by subtracting the result of the 44 kN run from the 178 kN run, thereby removing the effect of the rail, ballast, and subballast. The smooth trend in this figure suggests that the subgrade stiffness does not change much in this section, except for a soft spot at 3400 m. The TLV is intended to locate track with low vertical support and to differentiate whether the cause is a result of the ballast or subgrade by comparing the contact deflection result with the deflection observed under the 178 kN loading (Li et al., 2004).

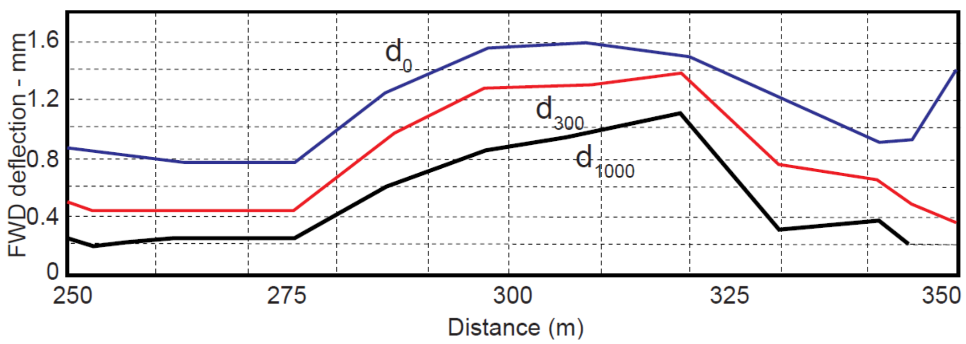

Figure 2-10. The deflection observed in FWD tests over a section of track (*after* Brough et al., 2003)

Figure 2-11. Track Loading Vehicle (Li et al., 2004)

2.4.3. Rolling stiffness measurement system

Banverkethas (the Swedish National Rail Administration) developed a device to measure dynamic track stiffness called the Rolling Stiffness Measurement Vehicle (RSMV) (Berggren et al., 2006). The system excites the track through two oscillating masses and measures the force and acceleration response of the track. The static axle load of the RSMS is 180 kN and the maximum axle load amplitude is 60 kN. The system is also capable of performing measurements at high speed (up to 50 km/h) and detailed investigations at low speed (10 km/h).

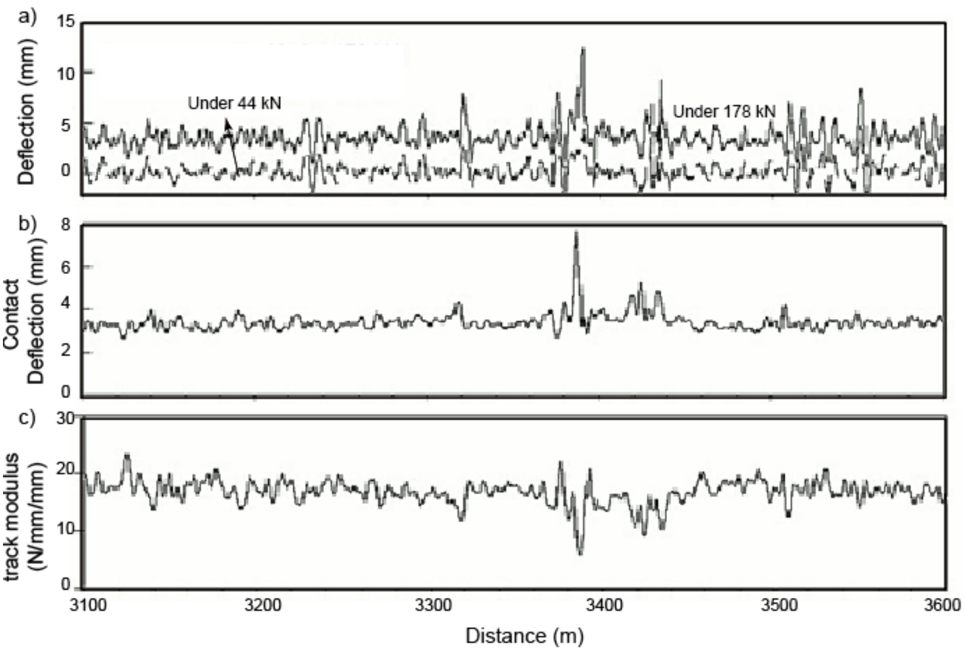

Figure 2-12. TLV test results: (a) deflection results for two separate tests of the same section, (b) contact deflection obtained from the difference between the results of the two runs, and (c) calculated track modulus (Li et al., 2004)

2.4.4. University of Nebraska rolling deflection system (MRail)

The University of Nebraska at Lincoln (UNL) in a collaborative project with the Federal Railroad Administration has developed a system to continuously measure the track vertical deflection (Norman, 2004; Norman et al., 2004; McVey et al., 2005; Farritor, 2006; McVey, 2006; Arnold et al., 2006, Greisen, 2010; Farritor and Fateh, 2013). This system is used in this study to measure the vertical rail deflections and is commonly referred to as the MRail system (or simply MRail). The original configuration of the MRail system consists of two enclosures over the rails on either side of the car, each featuring two line lasers and cameras (Figure 2-13).

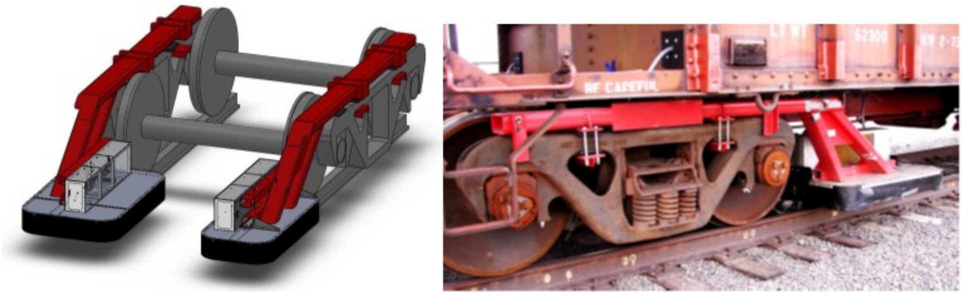

Figure 2-13. The UNL rolling deflection measurement technology (MRail system) shown attached to a three-piece truck. The laser and camera systems are housed at the inboard of the truck (Greisen, 2010).

The rail deflections measurements from this system are referred to as Y_{rel}, as they reflect the relative vertical deflection of the rail between where it contacts the wheel and at a distance of 1.22 m from the nearest wheel of the loaded truck. The distance of 1.22 m from the nearest wheel is a compromise between maximizing the distance from the wheel at which the measurement is taken and the increased vibration that occurs due to a longer and heavier beam (Norman, 2004).

The two lasers on top of each side of the rail continuously shine two lines on the head of the rail, with a camera taking pictures at a frequency of 90 frames per second. These pictures are then processed to provide a measurement of the distance (d) between the two lasers, which is a result of the distance between the enclosure and the rail; Y_{rel} is then calculated from d (Lu, 2008). The calculation of Y_{rel} assumes that the unloaded rail profile is straight and level. The newer version of the MRail system employs only one laser and uses its location within the picture to determine Y_{rel}. Both versions of the MRail system were tested during the course of this project, on different sites. The resulting datasets consist of latitude and longitude coordinates of the measurements, the estimated milepost (MP), and Y_{rel} values for the right and left rail at 0.305 m (1 ft) intervals. The distance along the rail between consecutive measurements is a result of the train speed. For example, measurements are taken at a 150 mm spacing when the train is traveling at a speed of 65 km/h. The recorded data are post-processed to provide a Y_{rel} value every 0.305 m so as to be comparable to other datasets (e.g., track geometry) collected by the railway industry. The accuracy of the Y_{rel} measurements has been evaluated by comparing the trackside measurements and linear variable differential transformer (LVDT) results with sensor measurements.

The sensor measurements during testing showed that the system is able to measure the correct trend in track stiffness (Norman, 2004). The repeatability of the measurements has been evaluated by conducting several runs over a small section of track. Statistical analysis of measurements show that all data fall within one standard deviation of the mean (McVey, 2006). Repeated tests over the same section of track at different speeds (33 and 77 km/h) show that, for most of the section, the measurements are repeatable. Exceptions are the location of defects and joints where an increase of up to 25% has been observed with increasing train speed; this was attributed to increased train dynamic (Lu, 2008; Griesen, 2010).

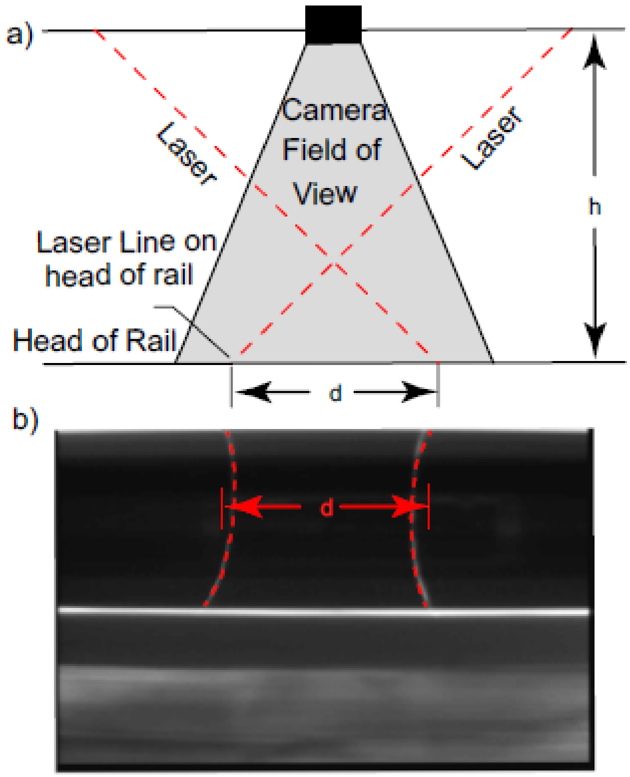

Figure 2-14. (a) Schematic showing the use of crossed lasers and camera to measure changes in distance between the laser system and the head of the rail 'd' and (b) a typical test image taken by an MRail camera (*after* Lu, 2008)

2.5. Remedial measures to improve track performance at soft subgrade locations

The railway industry adopts different practices to mitigate track constructed on soft subgrade. These methods can be divided into two categories. The first category is for methods that directly target the subgrade to improve its strength characteristics. These methods include the addition of admixture stabilization, geosynthetics, and piles. The second category are those remediation methods that improve the performance of track without directly addressing the subgrade. Tamping, ballast cleaning, ballast lifting, and the installation of stiffer ties and rails are among the methods that fall into the second category. Though the improved performance resulting from the direct methods lasts longer than indirect methods, the use of indirect methods is more common as they are less expensive and can be completed while the track is still in service. This section presents a brief explanation of these methods and the mechanism through which they improve track performance.

2.5.1. Direct subgrade remediation methods

The subgrade modulus is a major factor in the performance of track structures. By increasing the subgrade modulus by a factor of ten, an increase in the track modulus by a factor of eight can be achieved (Selig & Li, 1994). Therefore, improving the subgrade strength can significantly improve the track quality.

2.5.1.1. Admixture stabilization

Soil properties can be improved by mixing in other materials such as Portland cement, lime, bitumen, and fly ash (Selig & Waters, 1994). Mixing operations need special equipment to break up the soil and mix it with additives. In experimental studies by Hebib et al. (2003, 2004), the compressive strength of a stabilized material formed by mixing peat with cement was significantly greater than that of the original peat. In addition, the calculated settlement of the cement-treated peat was 28% lower than for peat that was not treated. Admixture stabilization is also a common treatment for expansive soil problems (Li et al., 2015).

2.5.1.2. Geosynthetics applications

Geosynthetics may perform the following functions in new track construction or rehabilitation: separation of materials with different particle size distribution, filtration, drainage, and soil reinforcement. In railroad construction, geosynthetics may be installed within or beneath the ballast or subballast layers.

The use of geogrids, as a form of geosynthetics, is becoming common practice in the industry. There are two design configurations to use geogrid in a railway track: 1) stabilizing ballast by installing geogrid at ballast and subballast interface and 2) improving bearing capacity of soft subgrade through installing the geogrid at subgrade and subballast interface. This increases track stiffness and reduces the deflections under axle loading (Selig & Waters, 1994; Göbel et al., 1994; Shin et al., 2002; Fernandes et al., 2008; Indraratna et al., 2006; Indraratna & Nimbalkar, 2013) and has also been shown to reduce the long-term settlement of the track structure under cyclic loading (Bathurst & Raymond, 1987; Raymond, 2002; Brown et al., 2007, Shin et al., 2002; Indraratna et al., 2006; Indraratna & Nimbalkar, 2013). Consequently, there will be less ballast disturbance and breakdown, leading to a lower maintenance frequency.

Field observations confirm that a geocomposite layer improves track performance through reduced vertical deformations when ideally placed at the ballast-capping interface (Indraratna et al., 2011). The results of a study conducted by British Railway Research suggest that the introduction of geogrid reinforcement into the ballast over a soft subgrade can provide a performance almost equal to a railway placed on a firm foundation (Tensar, 2006). Another field study conducted by the TTCI shows that introducing geogrid in a section of track with chronic ballast pocket issues improves the overall performance of the track and reduces the maintenance frequency from weekly to yearly (Bayse et al., 2015).

2.5.1.3. Piles and soil nailing

Pile foundations are a common remedial method usually adopted in situations in which the soil has a low bearing capacity and when a proper bearing stratum is not available at shallow depth. Using piles under railway embankment has shown the effectiveness of this method to transfer the loads from heavy axle loads to deep firmer layers. The observed field data over a peat foundation suggest that using the piles can reduce the induced pore pressure by 30% at the top of the peat layer and up to nearly 100% at the base of the peat (Hendry et al., 2011).
Soil nailing is another method of improving soft subgrade that can be performed with minimum disturbance. Soil nails can be driven into the subgrade at different inclinations from the space between the sleepers (Konstantelias et al., 2003). Soil nailing has been used to improve the track quality at a test embankment consisting of soft clay at the Transportation Technology center site and proved to be a method with fast installation.

2.5.2. Indirect methods

Indirect methods improve soft subgrade sections via different mechanisms, such as keeping water away from the subgrade, eliminating the impact loads that result from a rough profile, and distributing the load over a wider area. The following section provides an explanation for each indirect method and their respective mechanisms.

2.5.2.1. Ballast cleaning

Ballast cleaning refers to an operation in which the entire ballast layer is removed from the track and is either screened or replaced with new ballast. Over a period of time, the ballast voids become progressively filled with fine particles (fouled). There could be different sources of the fine particles, such as ballast aggregate breakage upon being subjected to heavy axle load traffic, particles come from an underlying layer, and weathering including the effect of freeze-thaw. The first requirement to achieve substructure drainage is to keep the ballast clean enough to be able to drain water as fast as it enters. In addition, having the surface of the subballast and subgrade sloped towards the side of the track might help accelerate drainage. Fouling can produce either an increase or decrease in the plastic strain accumulation (elastic modulus of ballast layer) depending on the nature of the fouling materials (Selig & Waters, 1994). Mud will lubricate the ballast aggregates and make rearrangement easier whereas fine granular particles will increase ballast interlocking and make rearrangement more difficult. Despite all of the uncertainties about the actual effect of fouling on track performance, ballast cleaning is the main practice used by railways to improve track condition. According to the European Rail Research Institute (ERRI), ballast cleaning becomes appropriate when more than 30% of the ballast particles (by mass) are less than 22.4 mm (ERRI, 1994; Esveld, 2001), and is absolutely necessary when there is more than 40%. A ballast-cleaning machine removes the fouled ballast and puts cleaned and new ballast back in the track.

2.5.2.2. Tamping (surfacing)

Tamping is the most common method of maintenance used by railways to restore and maintain the profile and alignment of the track. Poor track geometry can dramatically increase the magnitude of dynamic loads, and thus tamping is thought to reduce the loading and thereby improve the subgrade performance. Surfaced (tamped) sections of track have historically demonstrated a lower rate of deterioration and overall improved performance (Lee Aitchison, personal communication, July 2013).

During tamping operations, a tie is lifted and the ballast under it is compacted until the track geometry parameters are reasonably close to their design values. Intervals between tamping typically range from 2 to 10 years, depending on the amount of traffic and degradation rate of the track geometry. Very poor sections of track may require tamping several times per year to maintain the serviceability of the track. There are some concerns associated with excessive tamping. Tamping has been shown to break down ballast and reduce the life of railway ties (Chrismer, 1990; Wright, 1983). Research carried out by the Association of American Railroads shows that 20 tamping squeezes can generate 6 to 10% fine material in a granite and limestone ballast, respectively (Chrismer, 1989).

Tamping also increases the risk of rail buckling on hot summer days, as the lateral resistance at the tie-ballast interface can be reduced by up to 60% after tamping (Selig and Waters, 1994).

2.5.2.3. Increasing the thickness of the granular layer

Granular layer thickness is defined as the combined thickness of the ballast and subballast layers between the subgrade surface and the tie bottom. Proper granular thickness is required to limit the traffic load-induced stresses on the subgrade. With growing trends of railroads toward heavier axle loads, adding more ballast under the rail and lifting the track is considered one of the most effective and economical approaches to upgrade track performance over soft subgrade sections. Increasing the granular layer thickness reduces subgrade stress through two effects. The first effect results from an increasing distance between the subgrade surface and the load andthe second effect results from the fact that, as the granular layer thickness is increased, traffic loads can be spread over a larger subgrade area because of an increased overall stiffness of the granular layer (Li and Selig, 1998).

2.5.2.4. Using heavy rails and longer ties and welding joints

Replacing the existing rail with heavier rail (larger cross section) and standard 2.6 m (8.5 ft) ties with longer 3.1 m (10 ft) ties is one practice considered by the rail industry to improve track performance over soft subgrade sections. The heavier rail is able to spread the load over more ties in the longitudinal direction. In addition, the heavier rail has a higher moment of inertia that limits the track deflection. Heavier rails are also more resistant to rail surface defects. The longer ties help to spread the load over a wider area perpendicular to the track. In addition, the thicker cross-section of longer ties improves the track modulus.

The purpose of using the continuously welded rail (CWR) is to reduce maintenance cost for both rails and rolling stock. It should be mentioned that even though the CWR provides a smooth surface for rolling stock, it increases the risk of buckling in hot weather and rail breakage in cold weather (Philips, 2014).

Even though there is an increasing trend to replace jointed rails with CWR, rail joints cannot be completely eliminated. Bolted joints are used to connect strings of CWR as well as in sharp curves where rapid wear may necessitate frequent rail replacement (Jeong et al., 2014).

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3. Chapter 3: Continuous Vertical Track Deflection Measurements to Map Subgrade Condition along a Railway Line: Methodology and Case Studies

3.1. Contribution of the Ph.D. candidate

All the work presented in this chapter, including the data collection, data processing, review of literature, analysis and discussion of the results and writing of the text is carried out by the Ph.D. candidate. As supervisor, Dr. M.T. Hendry has reviewed all parts of the work. This chapter is published with the following citation:

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3.2. Abstract

This paper presents the methodology developed for the use of continuous vertical track deflection measurements from a moving loaded rail car to map the subgrade condition along a railway line. This analysis of deflection data was developed following the collection of over 12000 km of measurements. It was evident from the collected data that unprocessed deflection measurements are heavily impacted by the track surface condition such as joints and geometry irregularities so as to obscure the deflections due to poor subgrade support. A filtering method was employed to minimize the influence of the surface condition which occur at short wavelengths and show the variations in track deflections due to changes in subgrade conditions which occur at longer wavelengths. The resulting processed data is compared with the geology over two study railway subdivisions to demonstrate that the processed vertical track deflection measurements are representative of the subgrade conditions. The filtering process, examples of the impact of track surface on the deflection measurements, and the limitations of the resulting data are also discussed.

3.3. Introduction

The demand for the transportation of goods by rail in North America has been increasing significantly over the last decade. Much of this demand is related to the transportation of bulk goods, such as crude oil and coal, often from remote locations. These remote locations are typically serviced by branch lines that may have low allowable axle loads, have only been subjected to light traffic, and traverse challenging terrain. This combination has resulted in the need to upgrade several branch lines by railway companies to handle mainline carloads of 130 tonnes (286 kips) and the expected increased volume of traffic.

The challenge in rehabilitating a branch line is limited knowledge about the history of the performance or construction of the track. Such lines are seldom as rigorously monitored and maintained as mainline tracks, and have often had multiple owners. This makes the suitability of the earthworks and track structures along these lines for increased axle loads difficult to assess. Of particular concern is that these secondary lines may have sections of track with a soft subgrade. Such conditions typically contribute to the degradation of track geometry, and the subsequent development of track defects (Esveld, 2001; Ebersöhn *et al.*, 1993; Cai *et al.*, 1994; Sussmann *et al.*, 2001). Additional concerns have been raised with respect to the increased stresses generated in the rail due to the poor foundation, combined with the older and lighter weight rail common on branch lines. The extent and relative stiffness of these poor foundations have not been comprehensively mapped and documented due to the lack of an economical and practical method to measure the stiffness of track structure over long distances. However, new technologies have been developed in Sweden (Berggren *et al.*, 2002) and at the University of Nebraska (MRail) (Greisen, 2010; McVey *et al.*, 2005) to measure the vertical track deflection from a moving loaded car.

The authors conducted extensive trials with the MRail system over 12000 km of track to map the variability in subgrade condition by using vertical track deflections measured beneath a loaded car. This paper presents a representative sample of these results, focusing on two branch lines, and the development of a subgrade mapping methodology, that can be used to identify sections of soft track foundation and the repeatability of these measurements. The methodology presented utilizes the MRail measurement system, but the approach and analysis are transferable to other systems that measure relative and total vertical track deflection under heavy axle loads from an instrumented rail vehicle.

3.4. Impact of subgrade conditions on vertical track deflection

Vertical track deflection is the net result of the deformation of all of the components that comprise the track, embankment and underlying soils. Several parametric studies have been conducted to quantify the impact of each of these components on the track deflection (Stewart and Selig, 1982; Stewart, 1985; Selig and Li, 1994; Shahu *et al.*, 1999; Shahin and Indraratna, 2006; Rose and Konduri, 2006). Though the approaches of these studies have varied, the conclusions have consistently shown that subgrade conditions have the largest influence on the magnitude of track deflection. With 80 to 90% of the track deflection being attributed to deformations in the subgrade over soft terrain (Selig and Li, 1994). This is a result of the thickness of the subgrade layer and low moduli for some subgrade materials. Given constant subgrade conditions, the embankment height and ballast depth become the major influences on track deflection (Li *et al.*, 2015). Thus, a change in vertical track deflection is primarily an indication of a change of subgrade condition.

The track deflection measurements presented within this paper do show that these parametric studies are idealized; specifically, in that they consider constant track conditions and do not show the localized impact of track components such as joints.

3.5. Vertical track deflection measurement system

Researchers have focused on developing automated measurement technologies to evaluate track modulus and stiffness using moving loaded vehicles (Li *et al.*, 2004; FRA, 2004; Norman *et al.*,

2004) or vibrating masses (Wangqing *et al.*, 1997; Berggren *et al.*, 2002). The system used in this study to measure the vertical track deflections was originally developed at the University of Nebraska at Lincoln in collaboration with the Federal Railroad Administration (Norman, 2004; Norman *et al.*, 2004; McVey *et al.*, 2005; Farritor, 2006; McVey, 2006; Arnold *et al.*, 2006, Greisen, 2010; Farritor and Fateh, 2013) and is commonly referred to as the MRail system (or simply MRail). This system was selected in our study due to its availability and the simplicity of the measurement technology.

The original configuration of the MRail system consisted of two enclosures over the rails on either side of the car, each featuring two line lasers and cameras. The track deflections measurements from this system are referred to Y_{rel}, as they reflect the relative vertical deflection of the track between where it contacts the wheel and at a distance of 1.22 m from the nearest wheel of the loaded truck (**Figure 3-1**a). The distance of 1.22 m from the nearest wheel is a compromise between maximizing the distance from the wheel at which the measurement is taken and the increased vibration that occurs due to a longer and heavier beam (Norman, 2004). The two lasers on top of each side of the rail continuously shine two lines on the head of the rail, with a camera taking pictures at a frequency of 90 frames per second. These pictures are then processed to provide a measurement of distance (*d*) between the two lasers, which is a result of the distance between the enclosure and the rail; Y_{rel} is then calculated from *d* (Lu, 2008). The system is calibrated after installation to account for variations of truck and wheel geometry. The calculation of Y_{rel} assumes that the unloaded rail profile is straight and level. The newer version of the MRail system employs only one laser and uses its location within the picture to determine Y_{rel} (**Figure 3-1**b). Both versions of the MRail system were tested during the course of this project, on different sites. The resulting datasets consist of latitude and longitude coordinates of the measurements, the estimated milepost (MP), and Y_{rel} values for the right and left rail at 0.305 m (1 ft) intervals. As the camera samples at 90 frames per second, the distance between consecutive measurements is a result of the train speed. For example, measurements are taken at 200mm spacing when the train is traveling at a speed of 65 km/h. The recorded data are post-processed to provide a Y_{rel} value every 0.305 m so as to be comparable to other data sets (e.g., track geometry) collected by the railway industry.

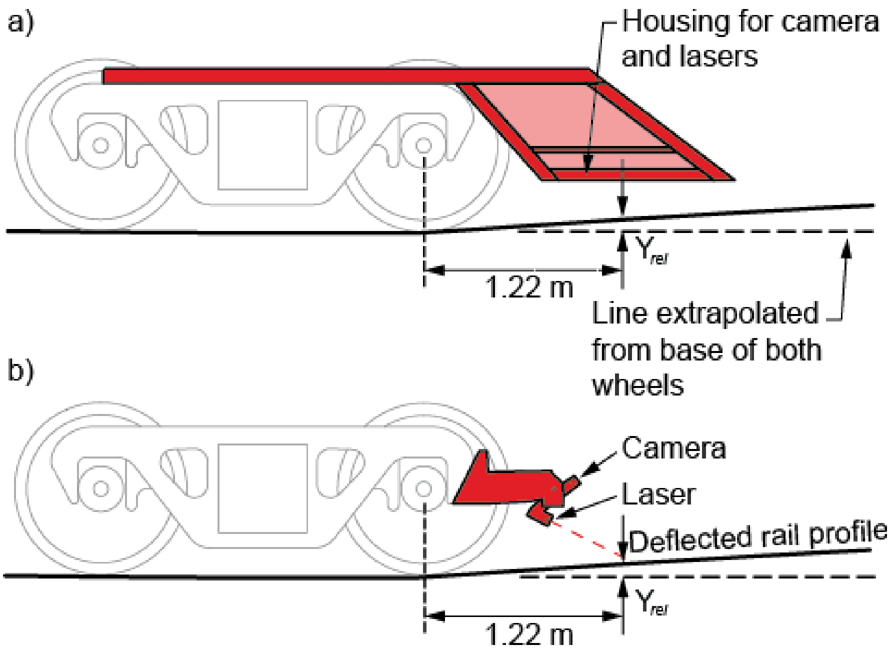

Figure 3-1. Configuration of (a) the original MRail system used for the measurements conducted over the PRCS and (b) the modified version of the MRail system installed on CN's gondola car and used for measurements on the LLBS.

The accuracy of the Y_{rel} measurements has been evaluated by the trackside measurements, which have used cameras and linear variable differential transducer (LVDT). The predicted Y_{rel} values from these methods were compared with the Y_{rel} values measured by the system and the accuracy of the measurements was shown to be within 10% (Farritor and Fateh, 2013). The sensor measurements during testing showed that the system is able to measure the correct trend in track stiffness (Norman, 2004). The repeatability of the measurements has been evaluated by conducting several runs over a small section of track. Statistical analysis of measurements showed that all data fall within one standard deviation from the mean (McVey, 2006). Repeated tests over the same section of track at different speeds (33 and 77 km/h) showed that, for most of the section, the measurements are repeatable. Exceptions are the location of defects and joints where an increase of up to 25% has been observed with increasing train speed; this was attributed to increased train dynamic (Lu, 2008; Griesen, 2010).

Previous work has suggested that the unprocessed Y_{rel} measurements can be used to identify track issues such as missing ties and unsupported rail joins that are not identifiable by other inspection methods including standard track geometry car (Farritor and Fateh, 2013). Hereafter, the (relative) vertical track deflections will be referred to as VTD, instead of Y_{rel}, as application of the methodology is not limited to the MRail measurement system.

3.6. The study sites

The two study sites examined in this paper are examples of branch lines on which significant increases of traffic and axle load are proposed, and for which an assessment of track performance is required. These two sites are Canadian National's (CN) Lac La Biche Subdivision (LLBS) in Northern Alberta, and Dakota, Minnesota and Eastern Railway's (DM&E) Pierre to Rapid City Subdivision (PRCS) in South Dakota. The track structure on both of these lines consists of 49.6 kg/m (100 lb/yd) jointed rail and wooden ties. The datasets collected over the LLBS and PRCS were used to understand the measurements and how they are impacted by the track features as well as to determine if VTD measurements could be used to map the subgrade condition.

The original version of the MRail system shown in **Figure 3-1**a, with the car loaded to 119.3 tonnes (263 kips) with axle loads of 29.9± 1.5 tonnes, was used to test the PRCS in April 2012 and July 2014. The newer MRail system shown in **Figure 3-1**b was installed on a CN ballast car loaded to 129.7 tonnes (286 kips) and axle loads of 32± 1.8 tonnes used to test the LLBS in October of 2013. The instrumented car was placed into a revenue train and the data recorded at the range of operating. The average train speeds were approximately 40 km/h and 16 k/h for LLBS and PRCS, respectively.

3.6.1. The Lac La Biche Subdivision

The LLBS runs from CN's mainline, near Edmonton, to Fort McMurray, Alberta. CN reacquired the LLBS from a short-haul operator (Athabasca Northern Railway and Lakeland & Waterways Railway) in 2008. The terrain through Northern Alberta is covered with large areas of wetlands and organic soils (muskeg). Up to 120 km of the 396 km railway line is estimated to be constructed over very soft muskeg foundations (Hendry *et al.*, 2013).

The track on the LLBS was considered to be in poor condition in 2007, and the previous operators were set to abandon the line. Large sections of the railway line ran over thin layers of sand and gravel without a railway ballast material that met modern specification (T. Edwards, personal communication, 2009). Since reacquiring the line, CN has significantly invested in improvements to the structures to allow heavy trains to safely average 40 km/h between Fort McMurray and Edmonton (Bourgonje and Scott, 2011). The long-term objective is to increase traffic from the current rate of one to two trains per day up to ten trains per day. The goal of this investment is "*the pipeline on rails*" concept, which would allow trains consisting of 100 or more tank cars to deliver oil from oil production facilities in Fort McMurray to the Gulf of Mexico refineries in eight to ten days (Alberta Oil, 2009).

The terrain along the LLBS has low to moderate relief, with extensive flat-lying and poorly draining areas. The performance of the LLBS is mainly affected by large expanses of muskeg (swamp) areas, with track supported on peat; in between these muskeg areas the track is typically supported by medium plastic silts and clays. These poor subgrade conditions have made the LLBS challenging to maintain. A large and comprehensive investigation was completed in October of 2013 as part of CN's evaluation of the requirements for further upgrades (Roghani *et al.*, 2015). This investigation provided extensive information as to the types of track components, height of embankments, and subgrade types. For more information regarding this subdivision, please also read Appendix A.

3.6.2. The Pierre to Rapid City Subdivision

The PRCS is 272 km of track that connects the cities of Pierre and Rapid City, South Dakota. This subdivision is operated by the Dakota, Minnesota and Eastern Railroad (DM&E). When the measurements reported here were taken, the DM&E was a subsidiary of Canadian Pacific (CP), which acquired the DM&E in 2007/08 with the intent to extend the railroad into the Powder River Basin and the nearby large coal mines. CP started extensive track bed upgrading with work conducted over the summer of 2012. The DM&E was purchased by short line operator Genesee and Wyoming Inc. in 2014. The PRCS is classified as 'excepted track', which is the lowest FRA classification, and carries a 16 km/h (10 mph) speed limit for freight trains; revenue passenger trains cannot use the track. The performance of the PRCS has been limited by challenging local subgrades including deposits from the Bad River as well as the Pierre formation (known as Pierre shale). The Pierre shales are notable as they weather at the surface to extremely soft high plastic clay (Searight, 1952).

3.7. Analysis of vertical track deflection measurements

Figure 3-2a shows unprocessed VTD data recorded on one side of the rail over a 32 km long section of the LLBS. This section of track contains both continuously welded and bolted rail segments. There is a clear difference between measurements taken on track with continuous welded rail (CWR) versus jointed rail.

Figure 3-2b shows the variation of the measurements on both sides of the rail over a 200 m subsection of this track. Peaks in the values that occur at a relatively constant 12 m interval on each side of the rail correspond to the space between joints in the track; the negative values measured after the large peaks correspond to the joints. From the configuration of the system, a negative VTD value implies that the rail is rising above its original position. This is not a true upward movement but the result of a sharp localized reduction of track support over a very short distance at the joint in the rails. This inconsistency shows that the magnitude of VTD from this system is strongly influenced by the track surface conditions immediately beneath the measurement location as well as both axles on the truck. Localized soft locations result in a change in the inclination of the truck with an exaggerated VTD when the first axle of the truck reaches the soft location (**Figure 3-3**a) and a negative VTD when the second axle reaches the soft location (**Figure 3-3**b). These high and negative values were also recorded on CWR over bridge abutments. Examination of data recorded along the LLBS and PRCS shows that VTD collected at locations of joints or inconsistent track geometry are highly variable. This variability adds another factor to the parameters that influence the VTD and that is the impact from track surface condition which does not allow the interpretation of VTD for mapping the subgrade condition. Thus, the hypothesis was developed that the impact of surface condition could be filtered out, with the resulting processed data providing a measure of the substructure and subgrade condition beneath the track.

3.7.1. Impact of track surface condition measured deflections

In order to assess the extent of impact that the track surface condition has on the VTD from, an analysis of the magnitude and wavelengths of the measurements was performed to examine their composition. This analysis was carried out with a Fast Fourier Transform (FFT) using MATLAB (2015). **Figure 3-4** presents the results of FFT analyses conducted on two 8-km long samples of VTD measurements from the LLBS (jointed rail, **Figure 3-4**a; CWR, **Figure 3-4**b). The amplitudes of both FFT plots are normalized to the maximum amplitude obtained from the FFT analysis of the VTD measurements from jointed rail. The major contributing amplitudes for the dataset in **Figure 3-4**a occur at wavelengths between 5.5 to 6.5 m for bolted joints. From **Figure 3-2**, this distance is the average space between two staggered joints on alternate rail, which is half of the 11.9 m standard length rail. For both the LLBS jointed rail and CWR, the spectrum of higher magnitude and shorter wavelengths extends up to wavelengths of approximately 20 m; this is followed by a gap that extends up to wavelengths of 100 m (**Figure 3-4**a and b). The amplitudes of the short wavelength variations in VTD obscure the contribution of the longer wavelengths. The wavelengths greater than 100 m are of a scale more consistent with the length of features resulting from variations of earth structures, geology, and terrain. This spectrum of wavelength may vary from site to site. FFT analysis of data from PRCS (data not shown) shows similar results to that from the LLBS analysis.

3.7.2. Deriving a subgrade index from track deflections

To minimize the influence of surface imperfections, the VTD data were filtered to remove lower wavelength data resulting from surface structure conditions. This was accomplished with a moving average, a method selected for its simplicity and ability to reduce noise while retaining sharp step responses (Smith, 1997). The result is a filter version of the VTD data set (VTD_{sub}) that is a measure of the vertical track deflection that is predominantly a result of subgrade conditions (Equation 3-1). The filter averages the points over a length of track on either side of each datum, as follows:

$$VTD_{sub}(i) = \frac{0.305}{L} \sum_{-L/(2)(0.305)}^{L/(2)(0.305)} \mathrm{VTD}[i+j] \ , \qquad \text{Equation 3-1}$$

where L (in m) is a threshold length such that lower wavelengths will be filtered out. Features that occur at lengths shorter than L will be removed, and sharp transitions will be spread over a distance of $2L$. The required resolution, and thus L, is a result of the length of features of interest. The maximum resolution of subgrade features was obtained using an L of 20 m. The effect of the selection of L will be demonstrated in subsequent analyses.

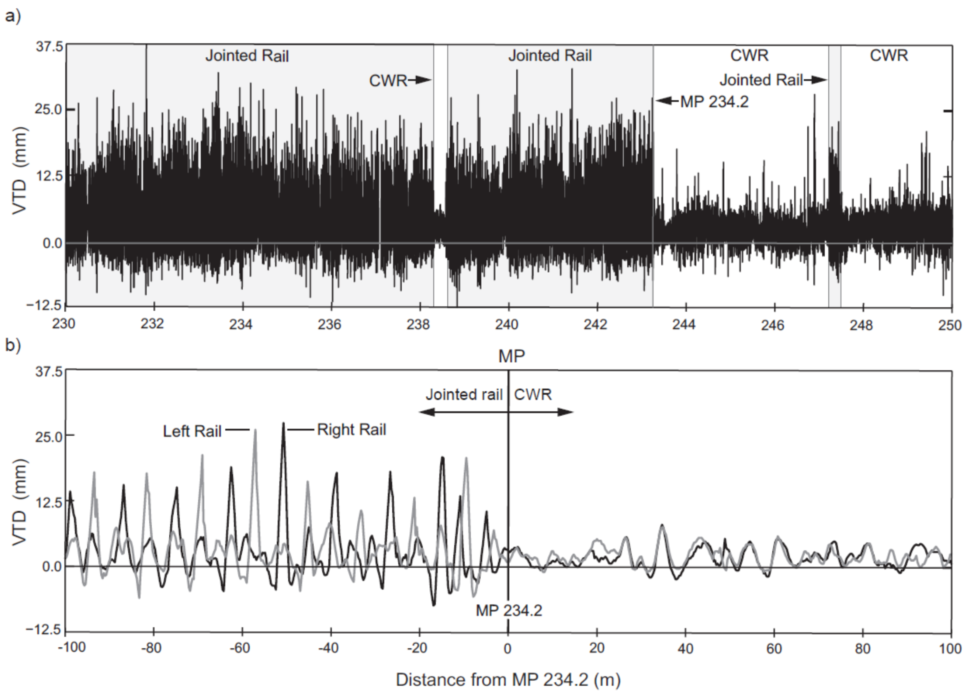

Figure 3-2. Plot of (a) VTD data recorded over a 20-mile section of the LLBS that contains both jointed and CWR and (b) a 200 m long section of this VTD data showing both the left and right rail and the change from jointed to CWR near MP 234.3.

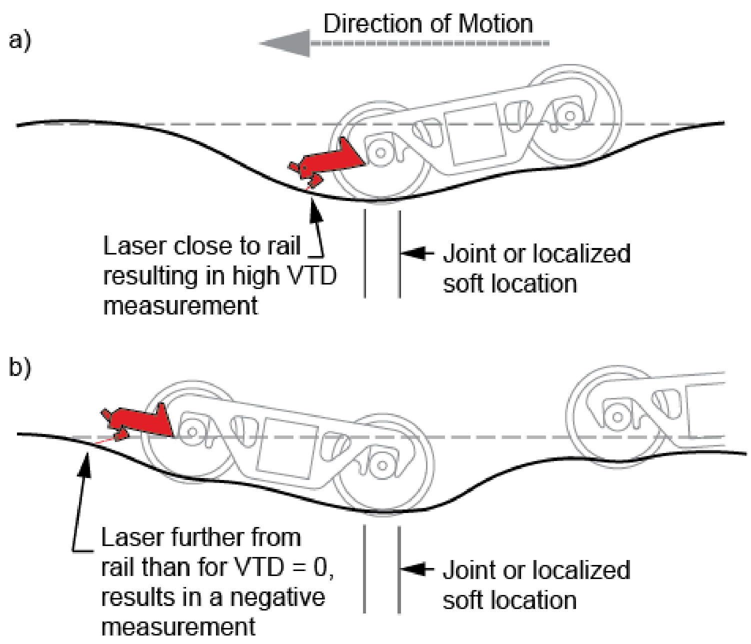

Figure 3-3. Illustrations of the effect of joints or localized soft locations on the measured VTD. This includes (a) exaggerated deflection measurements when the nearest axle is over the joint and (b) a negative deflection measurement when the furthest axle (on the same truck as the MRail system) is over the joint.

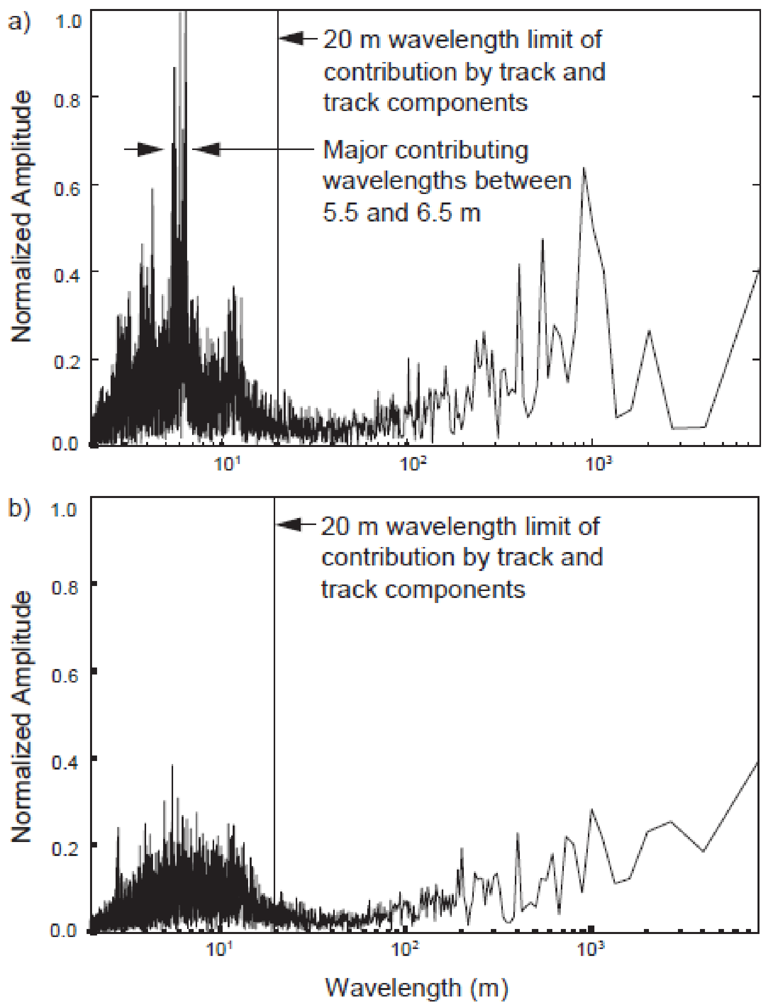

Figure 3-4. The results of the FFT analysis of VTD measurements plotted for (a) jointed rail and (b) CWR from the LLBS. The amplitudes of both plots are normalized to the maximum jointed rail.

The VTD_{sub} measurements were compared with subgrade conditions at the two study sites to evaluate the filtering procedure and its suitability to identify soft subgrade locations.

3.7.3. Correlation of VTD_{sub} to substructure and subgrade conditions

The highly contrasting track deflection that results from muskeg formations on the LLBS provides the clearest demonstration of the ability of the VTD_{sub} to show subgrade conditions. An example is shown in **Figure 3-5**, where **Figure 3-5**a shows the stratigraphy of an 800 m section from the LLBS determined during extensive investigation of the structures from surface mapping with ground penetrating radar (EBA Engineering Consultants Ltd., GPR survey, unpublished report,

2009) and shallow sampling (Roghani et al., 2015) and **Figure 3-5**b plots the VTD$_{sub}$ measured over this site and shows a clear correlation between the location and depth of the muskeg (see also **Figure 3-6**). Similar analyses of sections of track over bridges clearly show the increased stiffness of these structures (**Figure 3-7**). This correlation is also evident over longer sections of track; **Figure 3-8** compares the VTD$_{sub}$ (**Figure 3-8**b) and the mapped extents of muskeg over a 19 km section of the LLBS (**Figure 3-8**a) (Andriashek, 2002; BGC Engineering Inc., Lac La Biche Subdivision Peat Assessment, unpublished report, 2008). The limits of the muskeg are clearly visible, and variations in VTD$_{sub}$ within the muskeg formation are correlated to the depth of the muskeg and the thickness of the embankment structures.

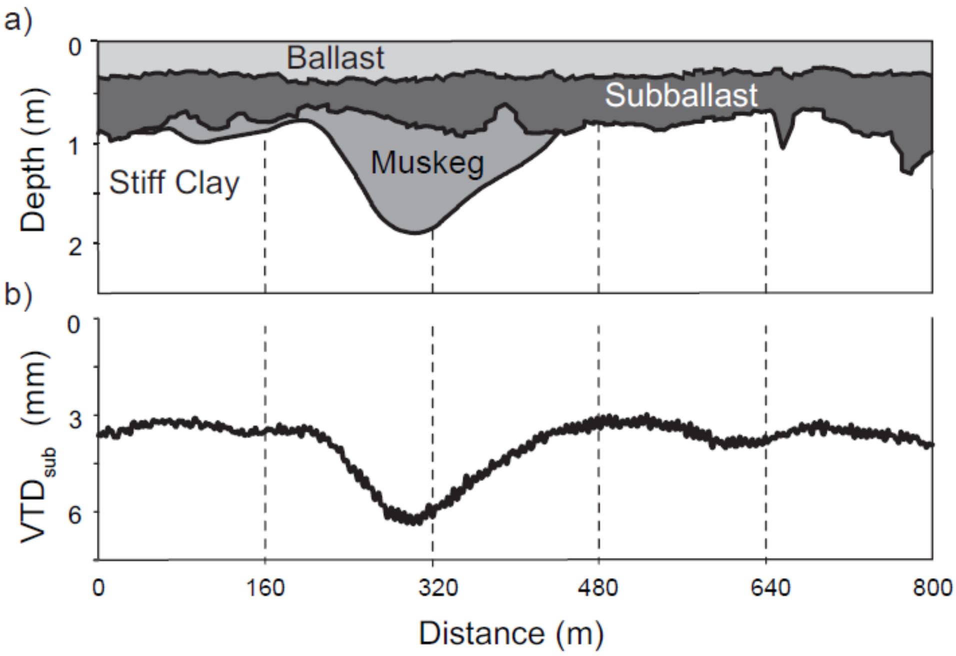

Figure 3-5. Variation of VTD$_{sub}$ data (calculated with a 20 m filter) over a section of track constructed on top of a section of embankment crossing over a ~450 m of a muskeg formation of varying depth.

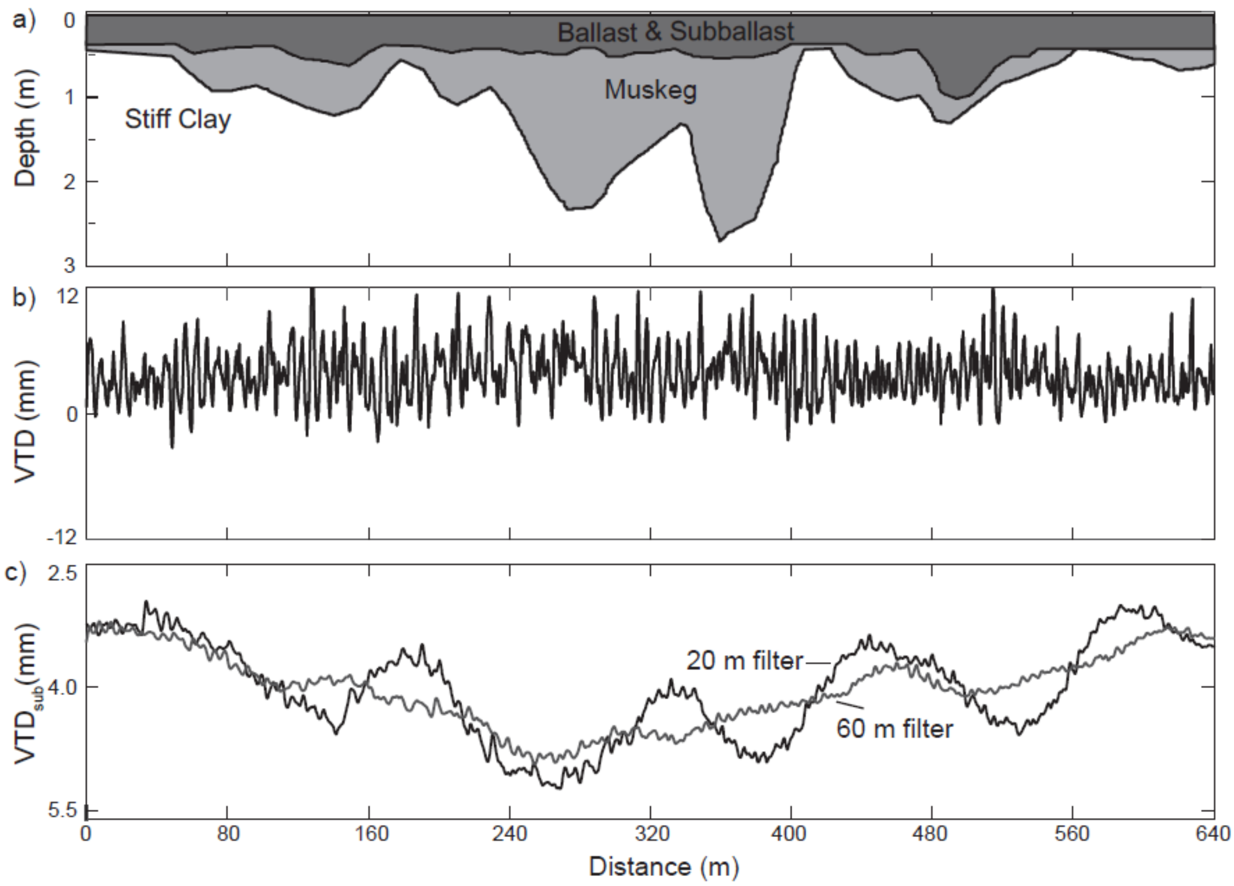

Figure 3-6. Variation of VTD$_{sub}$ data over track constructed on top of a section of embankment crossing over a ~640 m of a muskeg formation with varying depth: (a) depth of embankment and muskeg, (b) unprocessed VTD measurements, and (c) VTD$_{sub}$ values resulting from 20-m and 60-m filtering thresholds.

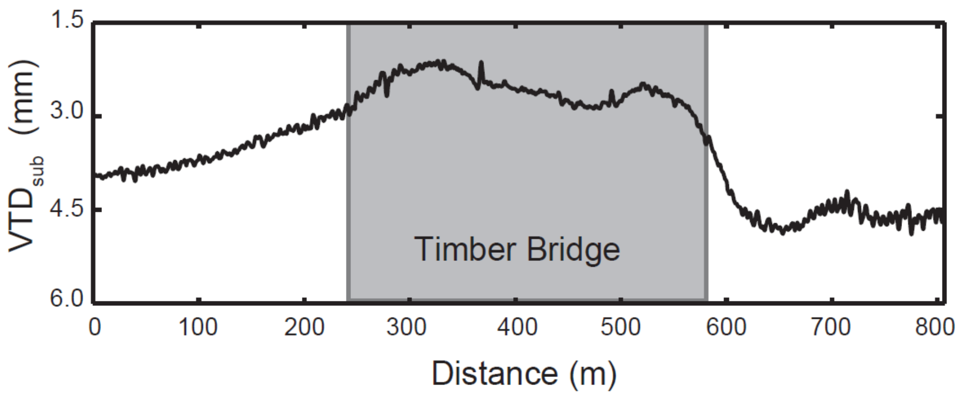

Figure 3-7. Plot showing gradual increase and decrease in the VTD$_{sub}$ measurements (calculated with a 20-m filter) as measured over a timber bridge with 50 spans and the transitions on either end.

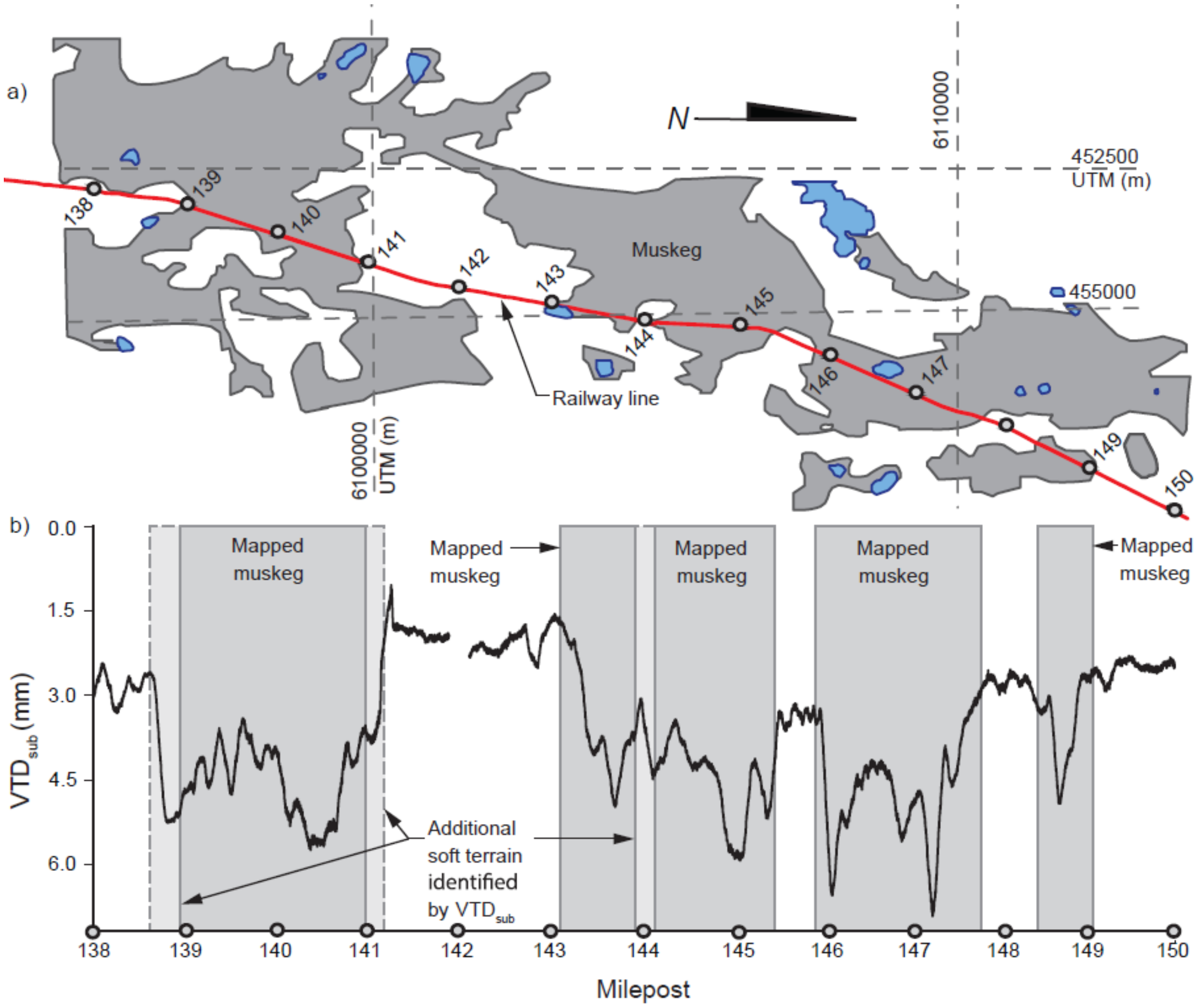

Figure 3-8. Comparison of the VTD$_{sub}$ data (calculated with a 20 m filter) with the locations of the muskeg terrain identified from existing surficial geological mapping in Andriashek (2002) and air photo interpretation from BGC Engineering Inc. (2008).

3.7.4. Correlation of VTD$_{sub}$ to subgrade conditions on a subdivision scale

The variations in VTD$_{sub}$ are best shown with measurements from the PRCS. The changes in VTD$_{sub}$ occur over longer distances; this allows plotting of the whole subdivision in to show the range of large variations that result from changes in the stiffness of the foundation materials. The VTD$_{sub}$ data presented in **Figure 3-9** were calculated with a 5-km filter to allow for the analysis at this scale, which removes the impact of local changes of track and earthworks construction. Overall, the VTD$_{sub}$ increases from 5.5 mm at the start (eastern limit) of the measurements at MP 500 to 4.1 mm at MP 637. Within this portion of the PRCS are several locations of notable increases in VTD_{sub} values, all of which are attributable to geological formations and softer foundation materials (high plastic alluvial deposits within valleys).

Starting from Pierre, the PRCS follows the Bad River and the South Fork of the Bad River to MP 571. The stiffness of the subgrades within this valley and the resulting VTD_{sub} data are variable, as the track was constructed on the edge of the valley to limit grades. Increases in VTD_{sub} within this valley occur at locations where the railroad crosses the bottom of the valley and the subgrade is comprised of highly plastic clay that was washed into these valleys and deposited as alluvium. These increases in VTD_{sub} values are evident at MP 540 and MP 524, and very evident near Pierre where the railroad crosses the wide valley at the fork of the Bad River (**Figure 3-9**). Aerial photographs from this area show evidence of farming, which suggests higher water content within the alluvial soils. A location of very high VTD_{sub} values is visible at MP 512 near the base of the river valley. This site was identified by DM&E and CP engineers as having notably poor conditions due to subgrade conditions and being in need of rehabilitation. Test pits conducted as part of this study revealed highly plastic clays, notable deformations beneath the track, and heaving at the toe of the embankment structures; combined, these factors resulted in the formation of a water retaining sub-structure beneath the railroad (**Figure 3-10**) (Li and Selig, 1998).

The PRCS measurements show two sections of the subdivision west of MP 512 with VTD_{sub} values approximately 50% greater than the surrounding terrain. At this location near the town of Wasta, the PRCS crosses the Cheyenne River Valley. The second feature occurs at MP 590, east of the town of Wall, where the PRCS traverses eroded badlands terrain just north of Badlands National Park. The major subgrade of this section of the PRCS is extensively eroded clay-rich soil (Searight, 1952).

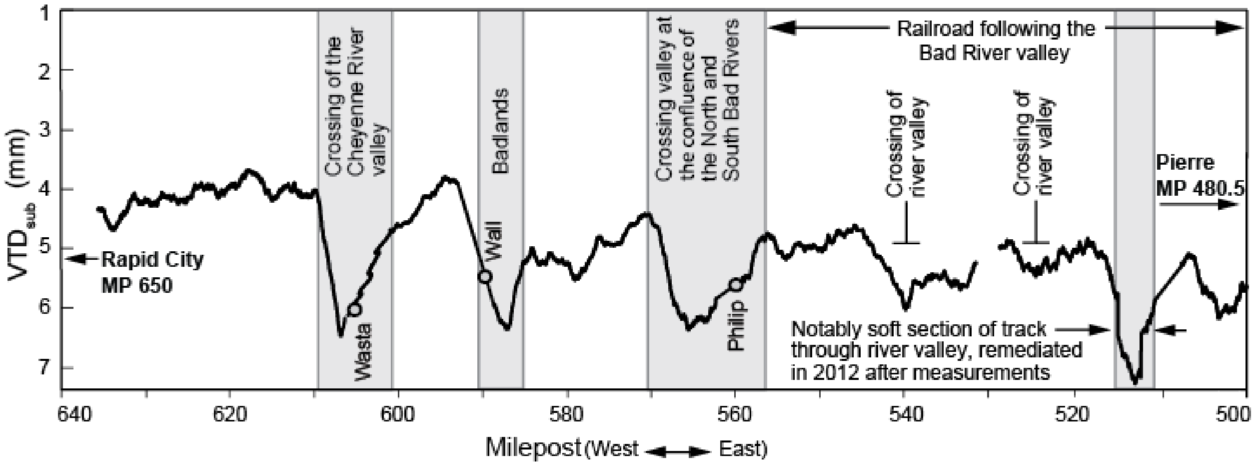

Figure 3-9. Plot showing the variation of VTD$_{sub}$ data (calculated with a 5-km filter) along the PRCS, the location of major notable features in the VTD$_{sub}$ dataset, and the corresponding geological features that result in the measured values.

Figure 3-10. Photograph of one of five test pits, showing evidence of heave in the high plastic clays (left-hand side) and standing water at the base of the fill material.

3.8. Repeatability of measurements

The 2012 and 2014 measurements of a 48 km section of the PRCS (MP 530 to MP 560) are compared in **Figure 3-11**. The VTD_{sub} data from 2014 run features higher values than the 2012 run due to the presence of a heavier car adjacent to the instrument truck. The MRail system was designed to be put 'in revenue service', which means within any train configuration that is constructed within a rail yard, but this does not result in repeatable measured VTD and the resulting VTD_{sub}. However, the 2012 and 2014 data show clearly similar trends in relative stiffness between the features, and demonstrate a strong, non-linear correlation with a coefficient of determination (R^2) of 0.75. The results from the PRCS show an increase of VTD_{sub} of approximately 25% over the softest sections of the subdivision.

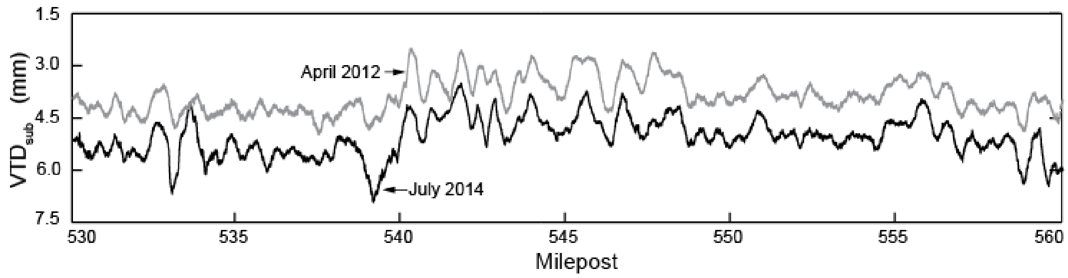

Figure 3-11. Plot comparing a 20-mile section of the VTD_{sub} dataset (calculated with a 500-m filter) from the 2012 and 2014 measurements on the PRCS.

3.9. Conclusions

A vertical track deflection measurement system was extensively tested to evaluate whether this measurement can be used for mapping the extent of soft foundations beneath a railway line. This study found that peak VTD measurements due to track irregularities and joints obscured variations in VTD due to changing subgrade support. A frequency domain analysis of the VTD data showed these particular VTD peaks occur at wavelengths less than 20 m. Thus, filtering out wavelengths less than 20 m reveals VTD features that are predominantly the result of subgrade condition and embankment construction. These VTD_{sub} values were compared to surficial geography, and two examples confirmed they are consistent with field conditions and represent meaningful data. This is valuable information for the assessment of the quality of the track structures that has not been available to the industry to date. The data were found to be very useful for evaluating the LLBS with respect to proposed increased axle loads and required capital upgrades.

Two notable limitations were identified with the use of vertical track deflection measurements. First, the filtering smoothed features shorter than the filtering threshold for a given analysis over a length equal to the filtering threshold. Thus, the interpretation of sharp changes in VTD requires judgement and knowledge of the site. Second, the load of the rail car adjacent to the measurement system greatly impacts the resulting measurements, and thus should be kept constant to ensure measurement repeatability and facilitate the comparison of subsequent measurements. Experience from this study indicates this is very difficult to achieve when the car equipped with the MRail system is shipped in revenue service.

3.10. Acknowledgement

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4. Chapter 4: Quantifying the impact of subgrade stiffness on track quality and the development of geometry defects

4.1. Contribution of the Ph.D. candidate

All the work presented in this chapter, which includes data processing, review of literature, analysis and discussion of the results and writing of the text is carried out by the Ph.D. candidate. As supervisor, Dr. M.T. Hendry has reviewed all parts of the work. This chapter is accepted (November 10, 2016) for publication at the journal of Transportation Engineering with the following citation:

Roghani, A. and Hendry, M.T. (2016). "Quantifying the impact of subgrade stiffness on track quality and the development of geometry defects" Journal of Transportation Engineering, American Society of Civil Engineering (ASCE).

4.2. Abstract

This paper presents the quantification of the impact of subgrade stiffness on the prevalence of track geometry defects and degradation of track quality indices (TQI). The data included in this study comes from two high traffic subdivisions (>50 GMT (gross million tonnes) /year) in Canada with total length of 800 km and consists of vertical track deflection (VTD) measurements and three years of track geometry measurements. The VTD measurements were used to derive two indices that represent the magnitude and variability of the subgrade stiffness.

An analysis of the data shows that the locations at which defects occur correspond to location with low modulus (higher VTD) and high variability of track modulus. A similar correlation is shown with track roughness represented by a TQI. However, the correlation with the spectrum of TQI calculated was found to be poor. This was attributed to maintenance activities carried out to improve track conditions. The correlation with the TQI greatly improved when arbitrary

thresholds were applied and TQI values above this were treated as geometry defects. These results show that the locations that have low modulus and a high variability in the modulus are those that are the most difficult to maintain, and at which maintenance is not always able to keep up with the degradation of track geometry. Thus, VTD measurements evaluate the underlying causes that result in the degradation of track conditions and allow for the identification of sections where poor track conditions are most likely to develop.

4.3. Introduction

Low track modulus and large changes in track modulus have been identified as a cause of an increased rate of the degradation of track geometry and the subsequent development of track geometry defects (Ebersöhn et al., 1993; Read et al., 1994; Cai et al., 1994; Sussmann et al., 2001; Esveld, 2001; Zarembaski & Palese, 2003; Davis et al., 2003). The rate of the development of irregularities in track geometry is important as track geometry defects are the second leading cause of derailments in both the United States of America and Canada (Liu et al., 2012; TSB, 2013). The influence of subgrade on track performance has been identified through localized field measurements, observations and extensive experience within the industry. However, it is difficult to make the case to increase track stiffness when there has not been a means to estimate the improvement in performance or the reduction of the probability of developing unsafe track conditions. To date there has been only a limited quantification of the impact of track modulus as there has been no practical means to measure the modulus or associated vertical track deflection (VTD) under heavy axle loads over long distances, until recently.

The authors have completed extensive trials (over 12 000 km of measurements) with a VTD measurement system that has become commercially available (Roghani & Hendry, 2016). The VTD measurements were found to be highly impacted by variations in the track surface condition, however, filtering out wavelengths less than 20 m resulted in VTD_{sub} which is a measure of the stiffness of the subgrade and the embankment construction (Roghani & Hendry, 2016). Obtaining

VTD_{sub} over extensive lengths of track has presented the opportunity to further investigate the impact of track modulus and its variation on the performance of track geometry. This paper presents an analysis of VTD_{sub} and track geometry records to quantify the impact of VTD and changes in VTD, and thus track modulus and variation in track modulus, on the degradation of geometry and the development of geometry defects. The 800 km of data used in this analysis comes from two high traffic subdivisions separated by thousands of kilometers, in different physiographic regions of Canada and with different subgrade types. The track structure on both of these subdivisions predominantly consists of continuously welded rail (CWR) and concrete ties. The first subdivision is located within the interior plains (AKA the Prairies) and the second subdivision within the Canadian Shield. This results in a large database with wide range of track quality and condition for the purpose of this analysis.

4.4. Measured data sets

4.4.1. Vertical track deflection measurements

The VTD measurements were recorded using an MRail rolling deflection measurement system that was developed at University of Nebraska at Lincoln in collaboration with the Federal Railroad Administration (FRA) (Norman, 2004; Norman et al., 2004; McVey et al., 2005; Farritor, 2006; McVey, 2006; Arnold et al., 2006, Lu, 2008; Greisen, 2010; Farritor & Fateh, 2013). The system consists of a laser and camera that measures the deflection of track at a distance of 1.22 m towards the center of the car from an inboard wheel, relative to a datum at the base of that wheel (**Figure 4-1**) VTD measurements were taken every 0.305 m (1 ft) along the track on both rails. This VTD system was used over the two subdivisions between May and July of 2015. The collected VTD measurements were processed to calculate VTD_{sub} using the methodology presented in Roghani & Hendry (2016). In both measurement runs, the MRail system was installed on a ballast car loaded to 129.7 tonnes and axle loads of 32 ± 1.8 tonnes. The measurement system was operated within revenue service and as a result the weight of the axles of the adjacent car or locomotive could not be specified, only measured. The variations in loading of the car, or locomotive, adjacent to the measurements system may change VTD measurements and the resulting VTD_{sub} (Roghani & Hendry, 2016). For the Prairie subdivision, the instrumented car was adjacent to a six axle locomotive with axle loads of 34.5± 0.3 tonnes, and for the Shield subdivision it was adjacent to a freight car with axle loads of 30.5 ± 1.5 tonnes.

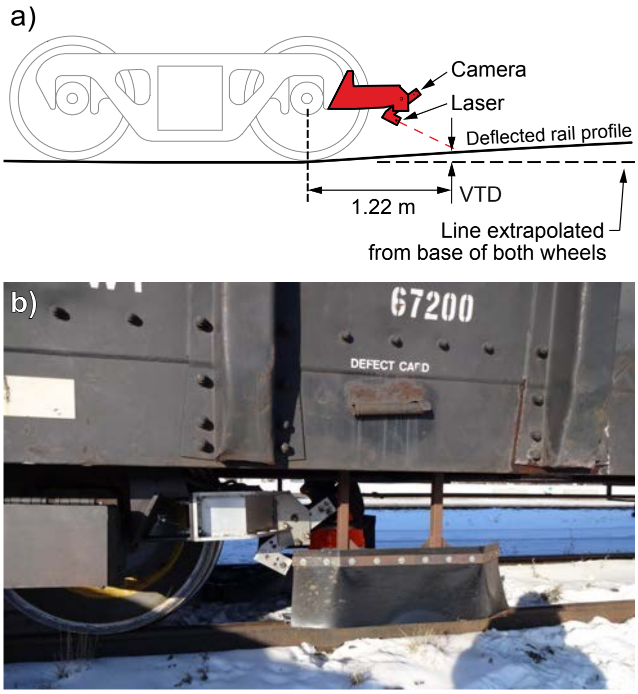

Figure 4-1. (a) The configuration of MRail system and the definition of VTD measurement (after Roghani and Hendry, 2016), and (b) a photograph of the MRail system installed on a car used to collect the VTD measurements for this study.

4.4.2. Track geometry measurements

Track geometry measurements are used by the railway industry to ensure that the shape of the rail allows for the safe passage of trains at the designated maximum speed of the track (AREMA, 2012). The ability to maintain operable track geometry is the primary function of the infrastructure beneath the track, and thus, the ability of these structures to maintain this geometry is the metric by which the subsequent analyses defines performance.

4.4.3. Track geometry measurements

The geometry measurements used in this analysis included gauge, alignment and the surface parameters including profile, crosslevel and warp. These geometry measurements are standardized and regulated (AREMA, 2012; FRA, 2007; Transport Canada (TC), 2011). Where, the gauge is the distance between two rails measured 16 mm below the top of rail with standard gauge equal to 1435.1 mm; the alignment is the horizontal deviation of the gauge side (inside) of the rail from a line subtended from two points 18.9 m apart on this surface measured at the midpoint of that line (AKA a mid-chord offset); the profile is the mid-chord offset measured vertically on the surface of the rail; the crosslevel is the elevation difference between both rails on a tangent track; and, the warp is the difference in crosslevel values between two points located 18.9 m (62 ft) or 9.5 m (31 ft) apart along the track. Geometry measurements were also taken every 0.305 m (1 ft) along the track.

4.4.3.1. Track geometry defects

A track geometry defect exists when the measured values of track geometry exceed threshold values set within regulations (FRA, 2007; TC, 2011; **Table 4-1**). These threshold values are defined based on an assigned class of track, where the class of track is defined so as to limit the speed of trains to match the condition of the track. According to both Transport Canada and the FRA, Class 1 has the lowest maximum track speed of 16 km/h (10 mph) and the highest geometry thresholds; and, Class 5 has the highest of 129 km/h (80 mph) and the lowest geometry thresholds. A section of track is maintained to meet the requirements of its assigned class, and thus the maximum allowable speed. Both subdivisions included within this study consisted primarily of Class 3 and 4 tracks. Short sections of Class 2 track exist within these subdivisions, but were excluded from this study.

Railway operators often refer to regulated defects as urgent defects. Priority defects are a second category with stringent thresholds defined by the operator. The threshold values used for priority defects in this study are presented in **Table 4-2**. The list of defects provided from the track geometry measurements consisted of three years of both priority and urgent defects from 15 different runs of track geometry car for each study subdivision. The dataset for urgent defects was too sparse (few and far between) for the analyses presented within this paper, thus, the priority defects and urgent defects were combined into a single dataset and are subsequently referred to collectively as defects.

A major focus of maintenance activities is to maintain the geometry of the tracks such that defects do not develop. Thus, the development of defects is a result of both the track conditions and the maintenance of the site. The locations of defects are locations at which the maintenance was not sufficient to maintain the track conditions to the standards of the railway operator.

Table 4-1. Regulated threshold values for defining geometry defects for freight service tracks (TC, 2011). *Note: the regulations provide these values in imperial units (mph, in and ft).*

Track classification	Maximum allowable speed km/h (mph)	Gauge not less than mm (in)	Gauge not more than mm (in)	Profile (surface) mm (in)	Crosslevel (Tangents and curves) mm (in)	Warp (over 18.9 m (62 ft) distance) mm (in)	Alignment (Tangent) mm (in)
Class 1	16 (10)	1416.1 (55.75)	1473.2 (58)	76.2 (3)	76.2 (3)	76.2 (3)	127.0 (5.0)
Class 2	40 (25)	1416.1 (55.75)	1466.9 (57.75)	69.9 (2.75)	50.8 (2)	57.2 (2.25)	76.2 (3)
Class 3	64 (40)	1422.4 (56)	1466.9 (57.75)	57.2 (2.25)	44.5 (1.75)	50.8 (2.0)	44.5 (1.75)
Class 4	97 (60)	1422.4 (56)	1460.5 (57.5)	50.8 (2)	31.8 (1.25)	44.5 (1.75)	38.1 (1.5)
Class 5	129 (80)	1422.4 (56)	1460.5 (57.5)	31.8 (1.25)	25.4 (1)	38.1 (1.5)	19.1 (0.75)

Table 4-2. Operator threshold values that define priority geometry defects for freight service tracks.

Note: the thresholds are defined by the operators in imperial units (mph, in and ft).

Track classification	Maximum allowable speed km/h (mph)	Gauge not less than mm (in)	Gauge not more than mm (in)	Profile (surface) mm (in)	Crosslevel (Tangents and curves) mm (in)	Warp (over 18.9 m (62 ft) distance) mm (in)	Alignment (Tangent) mm (in)
Class 1	16 (10)	1416.1 (55.75)	1463.7 (57.625)	50.8 (2)	25.4 (1)	57.2 (2.25)	95.3 (3.75)
Class 2	40 (25)	1416.1 (55.75)	1454.2 (57.25)	38.1 (1.5)	25.4 (1)	44.5 (1.75)	57.2 (2.25)
Class 3	64 (40)	1422.4 (56)	1454.2 (57.25)	32.7 (1.25)	25.4 (1)	38.1 (1.5)	34.9 (1.375)
Class 4	97 (60)	1422.4 (56)	1454.2 (57.25)	25.4 (1)	25.4 (1)	34.9 (1.375)	28.6 (1.125)
Class 5	129 (80)	1422.4 (56)	1454.2 (57.25)	19.1 (0.75)	17.5 (0.6875)	28.6 (1.125)	9.5 (0.375)

4.4.3.2. Track quality index

Track quality indices (TQI) are a common metric for track quality, and they are used to describe the variance, or roughness, of the available measures of geometry (Hyslip, 2002; El-Sibaie & Zhang, 2004; FRA, 2005; Berawi et al., 2010; Sadeghi & Askarinejad, 2010). These indices are useful in that they can be evaluated along the length of the track and provide a range of values, as opposed to defects that occur at discrete locations. TQI is not regulated within the North American railway industry but it has been suggested that it should be limited to reduce dynamic forces and thus the rate at which track components and rolling stock deteriorate (Zarembaski & Bonaventura, 2010). It is used within the following analysis to provide a representation of the roughness and condition along the length of the track.

The standard deviation of the geometric measures evaluated for a section of track provides a simple TQI that is as representative of track roughness as more complex formulations for TQI (ORE, 1981). A running standard deviation was used to calculate the TQI for the track profile (TQI_{PR}), gauge (TQI_{GA}), crosslevel (TQI_{CR}), and alignment (TQI_{AL}) (Equation 4-1). Where a higher TQI (standard deviation) shows the track to be rougher and infers that it is in poorer condition. These TQI were evaluated over 20 m running lengths of track so as to match the filter of the measured VTD data to which it will be compared (Roghani & Hendry, 2016).

$$TQI = \sqrt{\frac{\sum_{i=1}^{N}(x_i - \bar{x})^2}{N}}, \qquad \text{Equation 4-1}$$

Where, x_i is the deviation of geometry parameter measured at point i, N is the number of sequential measurement datum and $\bar{x}$ is the average of the data within the sample.

The assessment of performance based on TQI is complex, as the geometry of the track at any time is as much of a result of maintenance activities as it is of the performance of the infrastructure. Poorly performing structures which have undergone recent maintenance may have close to optimal geometry conditions; whereas, very competent track that has not required recent maintenance may have higher variations in geometry. The impact of maintenance is thus expected to obscure the trends in geometry that develop due to poorer performing infrastructure.

4.5. Evaluation of VTD based measurements

4.5.1. Development of threshold values for VTD_{sub} from AREMA standards

The published works and research regarding soft subgrades have quantified the condition of track using a track modulus (u), not VTD. Where, u is defined as the ratio of VTD and the pressure between base of the rail and the underlying ties and foundation, and is a measure of the stiffness of the structure (Cai et al., 1994). Parametric studies have shown u to be primarily influenced by the subgrade conditions (Stewart & Selig, 1982; Stewart, 1985; Selig & Li, 1994; Shahu et al., 1999; Shahin & Indraratna, 2006; Rose & Konduri, 2006). The numerical analysis conducted by Selig and Li (1994) concluded that changes in track modulus, where the modulus is < 28 MPa (4000 psi), results in substantial increase in track deflection, rail and tie bending stress, and subgrade stresses. Hay (1982) and the American Railway Engineering and Maintenance of Way Association (AREMA, 2012) manual suggest that 14 MPa (2000 psi) is minimum value of u required for a satisfactory performance of track. Similarly, Ahlf (1975) found through field observations that: a track modulus < 14 MPa resulted in track that required an exceptional amount of maintenance to maintain; a track modulus between 14 MPa and 28 MPa was average; and, a track with track modulus greater than 28 MPa was good. The terms satisfactory, poor, average and good used by AREMA (2012) and Ahlf (1975) are qualitative and describe the amount of maintenance that is required to maintain the track in an operational condition. Where, poorer performance requires more maintenance and monitoring to ensure operational conditions.

The analyses presented within this paper are conducted with VTD measurement and the thresholds for u are converted to VTD_{sub}, as VTD_{sub} can be measured and evaluated. This conversion was based on Fallah et al. (2016) which showed that the average u over a 20 m section of track with continuously welded rail (CWR) could be determined with the Winkler model and the VTD_{sub} measurement from this MRail system and axle loads. Thus, conversions for u to VTD_{sub} were developed for both loading conditions (adjacent car or locomotive) using the Winkler model; these modeled relationships between VTD_{sub} and u are compared in **Figure 4-2**. It is evident from **Figure 4-2** that there is very little difference (< 2%) between the VTD_{sub} values obtained when a locomotive is adjacent to the instrumented car as opposed to when a loaded rail car is adjacent. Thus, a single equation was developed to provide a conversion between u and VTD_{sub} for both of these loading conditions (Equation 4-2).

$$u = 348.8 \times VTD_{sub}^{-2.2} , \qquad \text{Equation 4-2}$$

Where, u is track modulus in MPa and VTD_{sub} is in mm. From **Figure 4-2** and Equation 4-2, VTD_{sub} > 4.4 mm is equivalent to the lower threshold of u < 14 MPa, and VTD_{sub} < 3.1 mm is equivalent to upper threshold of u > 28 MPa.

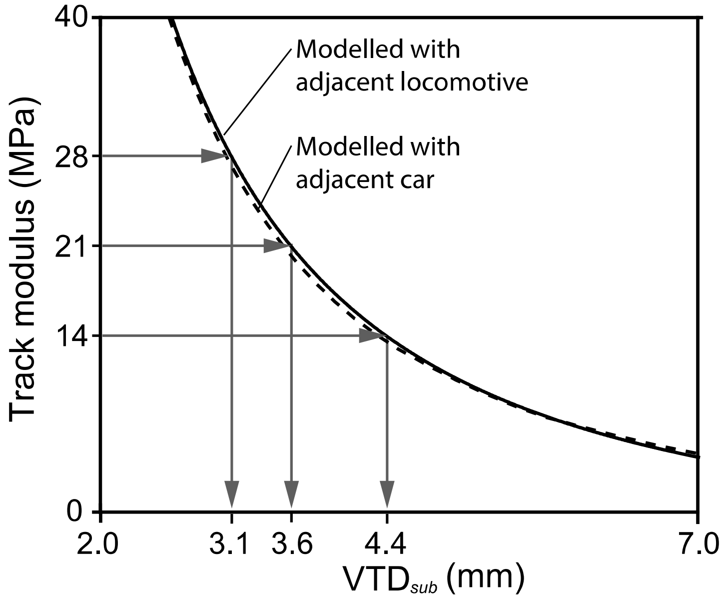

Figure 4-2. Plot showing the modeled relationship between VTD_{sub} and track modulus for the two loading condition under which measurements were obtained. *Note: the Prairies subdivision measurements were taken with a locomotive adjacent to the MRail system; and, the Canadian Shield with a loaded rail car adjacent to the MRail system.*

The distribution of VTD_{sub} from both study subdivisions is presented in **Figure 4-3**a, along with the threshold values for VTD_{sub} derived from the AREMA thresholds for u. Overall, VTD_{sub} has a normal distribution, where the mean (and median) of VTD_{sub} is 3.7 mm, and the standard deviation (σ) is 0.5 mm. From **Figure 4-3**a, the AREMA thresholds provide a reasonable agreement with the measured distributions, with 12% of the track classified as good and the threshold is 1.2σ below the mean, 78% as average, and 10% as poor and the threshold is 1.4σ above the mean. The Prairie subdivision shows a higher prevalence of poor track than the Canadian Shield subdivision, and thus softer subgrade conditions. The average range was subdivided at the mean so as to result in four categories, and increased resolution.

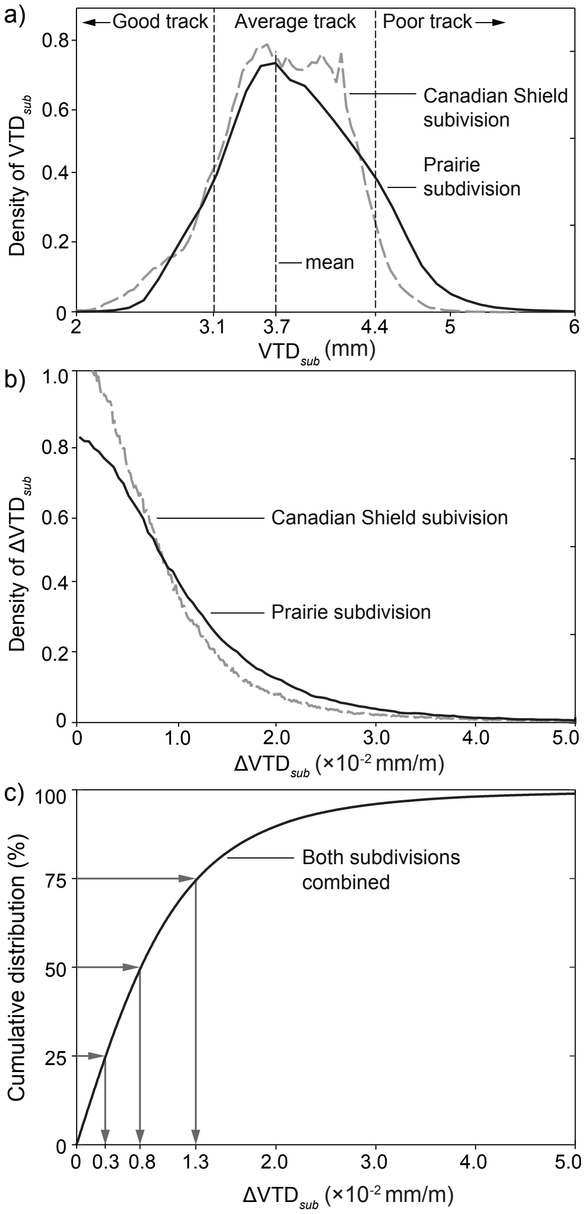

Figure 4-3. Presentation of the distribution of (a) VTD$_{sub}$ and (b) ΔVTD$_{sub}$ for the subdivision in the Prairies and the subdivision in the Canadian Shield; and, (c) the cumulative distribution of ΔVTD$_{sub}$ used to subdivide this dataset into quartiles (25% increments). *Note: Density is the normalized number such that the area under the curve is equal to one.*

4.5.2. Quantifying the change of VTD$_{sub}$

There is no precedence for the quantifying the change in modulus or the corresponding change in VTD, thus a metric was devised for this study. The slope of VTD$_{sub}$ versus distance plot was adopted as a simple and transparent metric to quantify change of track deflection (ΔVTD$_{sub}$) over a distance. Where, ΔVTD$_{sub}$ is calculated as the absolute value of secant slope of the VTD$_{sub}$, and distance (d) is the length of track over which this slope is evaluated (Equation 4-3). For this analysis d was set equal to 20 m to be consistent with the other metrics and filtering used for the analysis of the VTD measurements.

$$\Delta\mathrm{VTD}_{sub}\,(x) = \left|\mathrm{VTD}_{sub}\left(x + {d}/{2}\right) - \mathrm{VTD}_{sub}\left(x - {d}/{2}\right)\right|/d, \qquad \text{Equation 4-3}$$

The meaning of $\Delta\mathrm{VTD}_{sub}$ is demonstrated in **Figure 4-4**. Where, **Figure 4-4**a shows the stratigraphy of 800 m section of track, **Figure 4-4**b plots the VTD_{sub} measured over this track, and **Figure 4-4**c plots the $\Delta\mathrm{VTD}_{sub}$ calculated from the VTD_{sub}. An increase in $\Delta\mathrm{VTD}_{sub}$ is the result of an increase in slope in the VTD_{sub} plot and corresponds to a higher spatial change of VTD_{sub} and thus u.

The distribution of $\Delta\mathrm{VTD}_{sub}$, from both study subdivisions, is presented in **Figure 4-3**b. **Figure 4-3**b shows that the 99% of $\Delta\mathrm{VTD}_{sub}$ values vary between 0 to up to 0.05 mm/m within the two subdivisions evaluated. The folded-normal distribution of $\Delta\mathrm{VTD}_{sub}$ is a result of the use of an absolute value within Eq. 4-3, and results in a mode of 0 and a mean value of 0.009 mm/m. The Prairie subdivision shows a higher prevalence of high $\Delta\mathrm{VTD}_{sub}$. There are no thresholds that can be adopted to quantify the track conditions based on $\Delta\mathrm{VTD}_{sub}$, thus arbitrary values are imposed that divide the track into four that comprise equal lengths of track (quartiles). This division is based on the cumulative distribution of $\Delta\mathrm{VTD}_{sub}$ presented in **Figure 4-3**c; where, the 25, 50, and 75% quartile thresholds correspond to $\Delta\mathrm{VTD}_{sub}$ of 0.003, 0.008, and 0.013 mm/m.

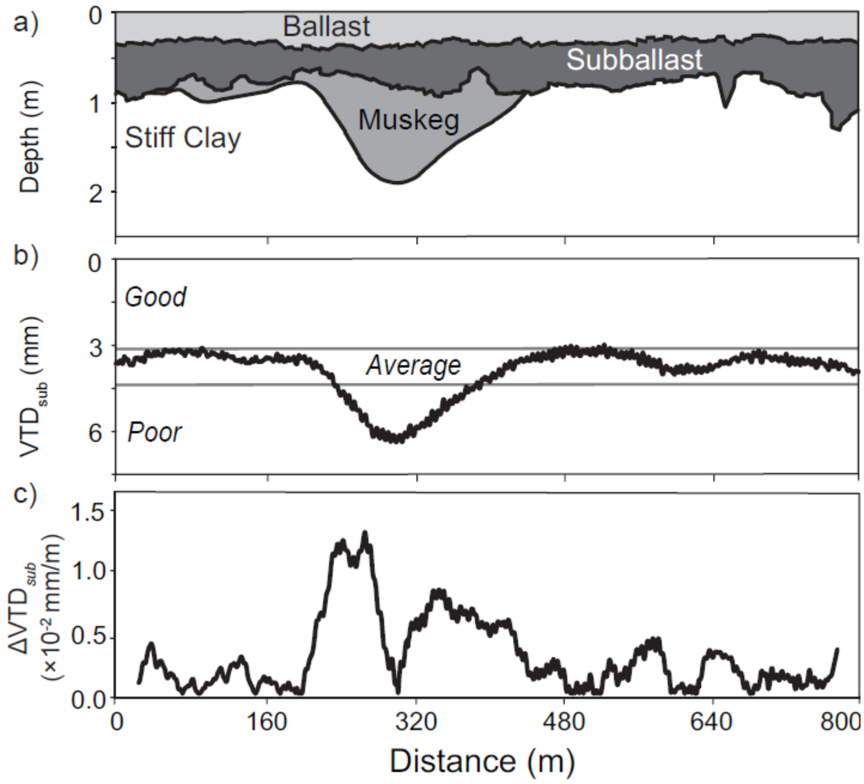

Figure 4-4. An example over an 800 m section of track with a section of soft muskeg subgrade to demonstrate the resulting VTD metrics. Where, (a) presents the composition of track substructure and shows the location of a soft muskeg section, (b) the VTD$_{sub}$ (*data from* Roghani and Hendry, 2016), and (c) the ΔVTD$_{sub}$.

4.6. Impact of VTD on track geometry

This section presents the quantification of the impact of the track deflection parameters, VTD$_{sub}$ and ΔVTD$_{sub}$, on the prevalence of defects and high values of TQI that imply poor performing track with an increased probability of developing unsafe track conditions.

4.6.1. VTD$_{sub}$, ΔVTD$_{sub}$ and the development of geometry defects

The defect data is comprised of discrete locations at which priority and urgent threshold values were exceeded. The comparison of the location of defects and the VTD measurements was based on the coordinates from the Global Positioning Systems (GPS) used for the measurements. Both the GPS system used for the track geometry and VTD measurements had a specified R95 (the radius of a circle centered at the true position, containing the position estimate with probability of

95%) of 3.7 m. Thus, the VTD_{sub} and ΔVTD_{sub} evaluated at the location of each identified defect was the average of the values measured within 7.4 m of the defect. These results were found to be insensitive to the variation of this 7.4 m window from 1 m to 10 m; this insensitivity is attributed to the 20 filtering applied to generate the VTD_{sub}.

This defect data was analyzed to determine the prevalence of defects per km of track within the different categories defined by the divisions of VTD_{sub} and ΔVTD_{sub}. Thus, the combined defects from both subdivisions were sorted into one of 16 categories based on which of the four divisions of VTD_{sub} and ΔVTD_{sub}. The number of defects within each of these categories was divided by the number of kilometers of track within each category so as to allow for a comparison of the intensity of the occurrence of defects generated by each classification of track. The results of this categorization are presented in **Figure 4-5** for Class 4 track, and **Figure 4-6** for Class 3 track. There were too few alignment defects to include in this analysis, with no Class 4 alignment defects and only 50 Class 3 alignment defects (or 3% of total number of defects).

Surface defects and gauge defects comprise 30% and 70% of the total number of Class 4 defects, respectively. **Figure 4-5**a and **Figure 4-6**a show a strong relationship between gauge defects and ΔVTD_{sub}, but not with VTD_{sub}, with the highest number of defects occurring with high ΔVTD_{sub}, and over the stiffest track (good VTD_{sub}). The authors hypothesize that this to be the result of higher dynamic loads that occur at transitions between differing track moduli, a mechanism suggested in Li et al., (2015), with the stiffest track providing less attenuation for impacts. The number of defects over the track with the most consistent VTD_{sub} (low ΔVTD_{sub}) suggests that there is only a very small contribution by factors not represented within this comparison. **Figure 4-5**b and **Figure 4-6**b show a strong relationship between surface defects and both VTD_{sub} and ΔVTD_{sub}, with the highest number of defects occurring with high ΔVTD_{sub} and the poor VTD_{sub}. These surface defects appear to be a result of both high deflections (low stiffness), and dynamic loads resulting from the changes in stiffness. The very small number of defects occurring where there are good VTD_{sub} and low ΔVTD_{sub} suggests that these two metrics describe the primary conditions that result in surface defects. These trends between gauge, surface defects, VTD_{sub} and ΔVTD_{sub} were found to be consistent between the two subdivisions with only slight variations in the magnitudes.

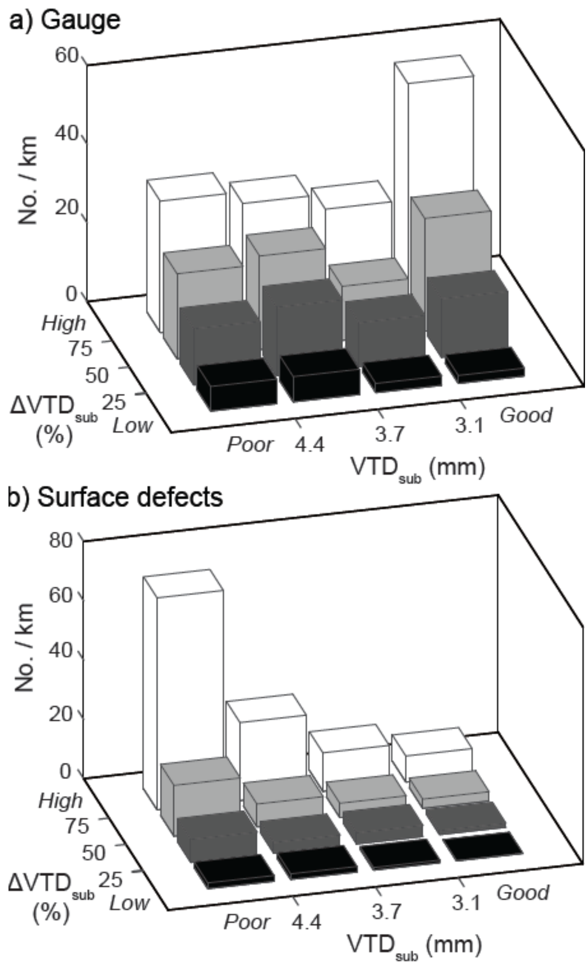

Figure 4-5. Plots of the distribution of the number of Class 4 (a) Gauge and (b) surface (warp, crosslevel, and profile) defects per km within the divisions of VTD_{sub} and ΔVTD_{sub}. These *defects* include both urgent and priority defects for both the subdivision in the Prairies and in the Canadian Shield.

The surface defects (**Figure 4-5**b) can be further divided into warp (**Figure 4-7**a), crosslevel (**Figure 4-7**b) and profile (**Figure 4-7**c) defects, each with a distribution within the VTD_{sub} and ΔVTD_{sub} classifications. These plots show that the highest frequency of warp, crosslevel, and profile defects occur with poor VTD_{sub} and high ΔVTD_{sub}. Both VTD_{sub} and ΔVTD_{sub} contribute to the generation of defects, though warp and profile defects show a greater impact of ΔVTD_{sub} (**Figure 4-7**a and **Figure 4-7**c).

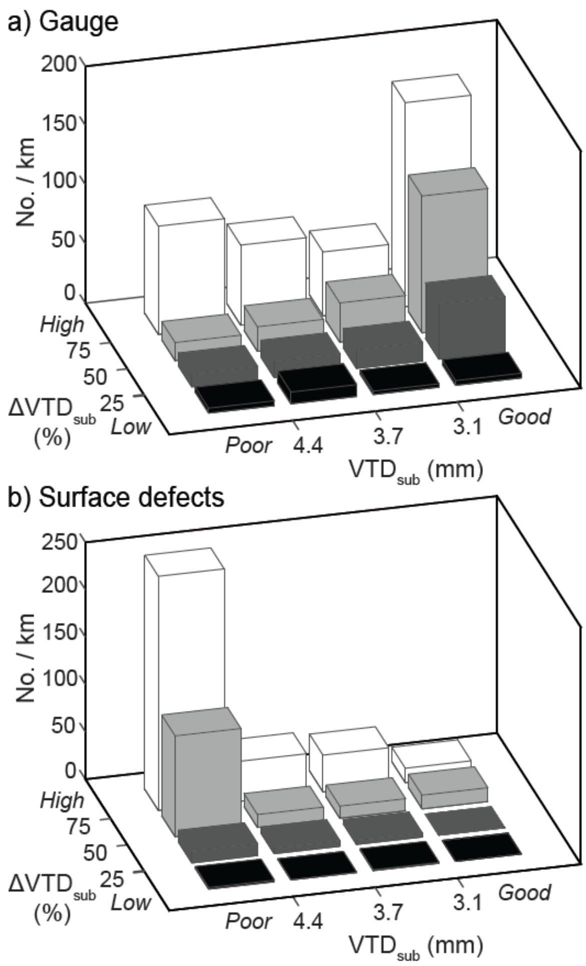

Figure 4-6. Plots of the distribution of the number of Class 3 (a) Gauge and (b) surface (warp, crosslevel, and profile) defects per km within the divisions of VTD_{sub} and ΔVTD_{sub}. These *defects* include both urgent and priority defects for both the subdivision in the Prairies and in the Canadian Shield.

Similar correlations were found between Class 3 warp, crosslevel, profile defects and VTD_{sub} and ΔVTD_{sub} from both subdivisions (**Figure 4-8**). The significance of poor VTD_{sub} and high ΔVTD_{sub} on development of surface defects is evident from all of the plots presented in **Figure 4-7**. The number of Class 4 warp, crosslevel, and profile defects per km generated at locations with ΔVTD_{sub} from the highest 25% was 2.6, 1.2, and 2.8 times that from the remaining 75% of the track, respectively (**Figure 4-7**a-c). Similarly, the number of Class 4 warp, crosslevel, and profile defects per km generated at locations with poor VTD_{sub} was 1.0, 1.7, and 1.7 times that from the remaining 82% of the track (**Figure 4-7**a-c).

4.6.2. VTD_{sub}, ΔVTD_{sub} and track geometry roughness

An examination of the VTD and TQI data was conducted to observe whether locations with higher VTD_{sub} and ΔVTD_{sub} correspond to higher local TQI values obtained from one measurement of track geometry. This examination showed that elevated TQI coincided with two cases of VTD: the first are locations with elevated VTD_{sub} and ΔVTD_{sub}; and the second are locations with elevated ΔVTD_{sub} but lower VTD_{sub}. An example of the first condition is presented in **Figure 4-9** which shows a 2 km section of track of poor track conditions ($VTD_{sub} \geq 4.4$ mm) with more competent track on both ends (**Figure 4-9**a) with high ΔVTD_{sub} at transitions and variations of stiffness (**Figure 4-9**b). TQI measures, excluding TQI_{GA}, increase within the section of poor VTD_{sub} (**Figure 4-9**c-d). Local peaks in TQI, including TQI_{GA}, are coincident with peaks in ΔVTD_{sub} (**Figure 4-9**c-d). An example of the second condition is presented in **Figure 4-10**, which shows an 800 m section of track of average to good track conditions based on VTD_{sub} (**Figure 4-10**a), with a 270 m section of track with high ΔVTD_{sub} (**Figure 4-10**b). TQI measures increase within the section of high ΔVTD_{sub} (**Figure 4-10**c-d), though the VTD_{sub} alone would suggest this to be a relatively competent structure.

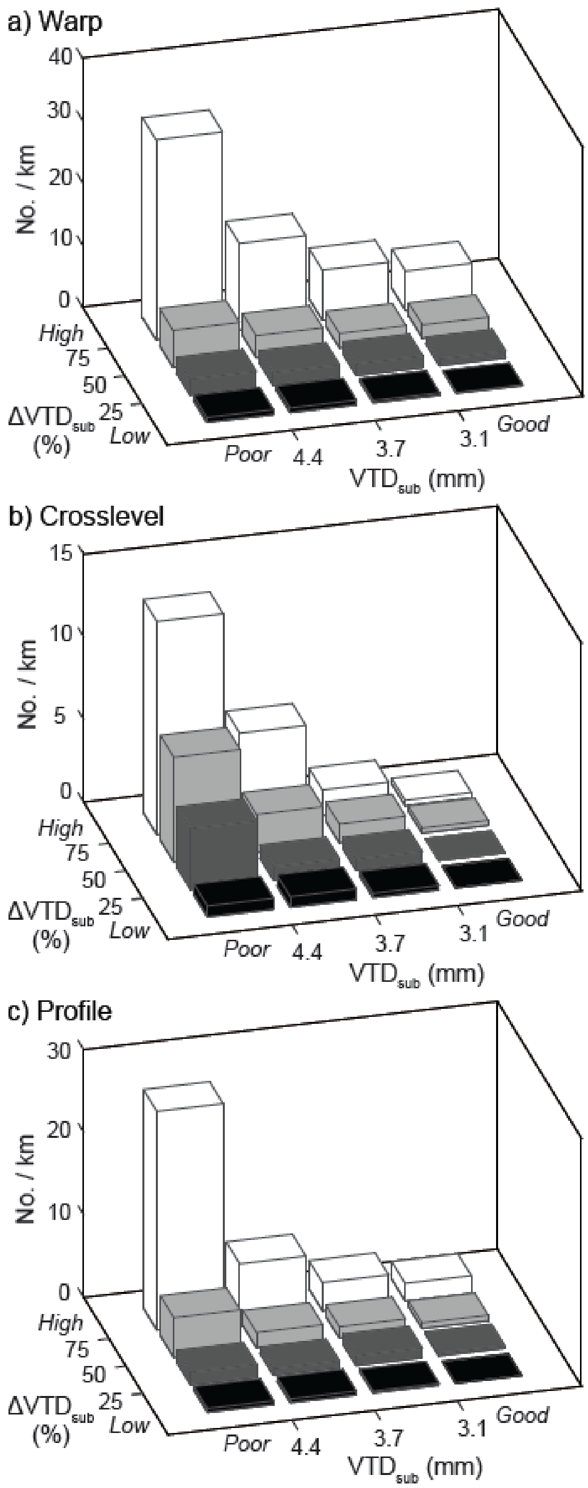

Figure 4-7. Plots of the distribution of the combined number of Class 4 surface defects per km divided into (a) warp, (b) crosslevel, and (c) profile defects within the divisions of VTD_{sub} and ΔVTD_{sub}. These *defects* include both urgent and priority defects for both the subdivision in the Prairies and in the Canadian Shield.

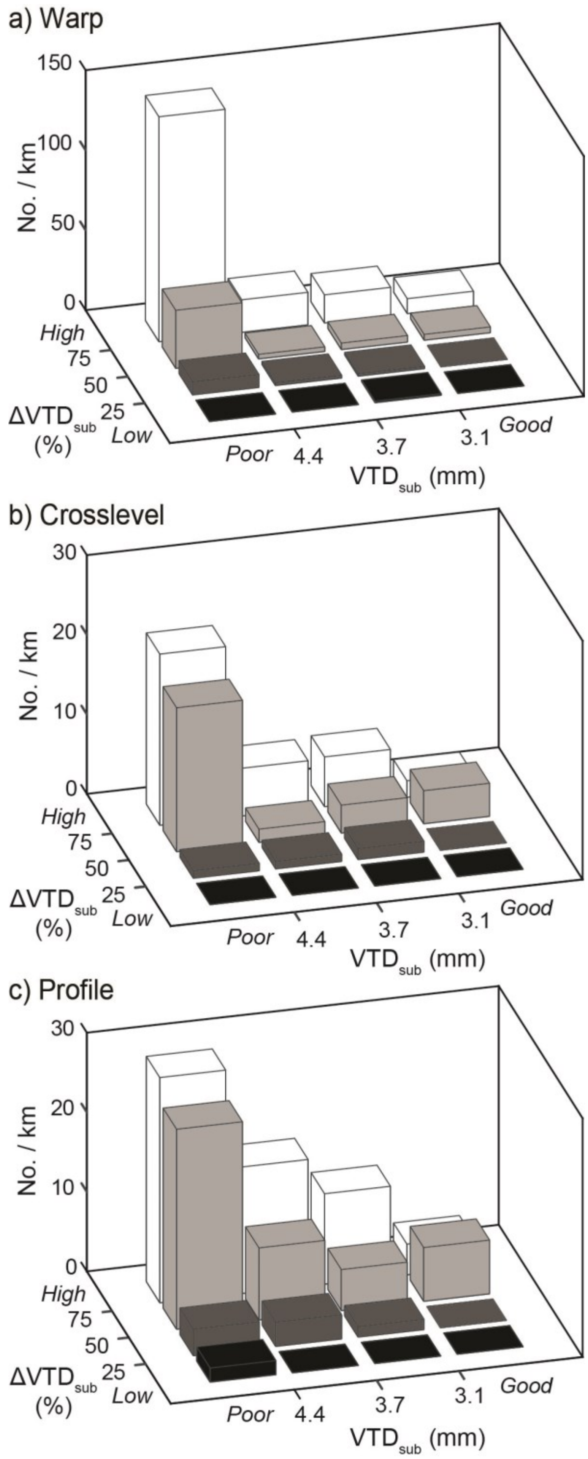

Figure 4-8. Plot of the distribution of the combined number of Class 3 surface defects per km divided into (a) warp, (b) crosslevel, and (c) profile defects within the divisions of VTD_{sub} and ΔVTD_{sub}. These *defects* include both urgent and priority defects for both the subdivision in the Prairies and in the Canadian Shield.

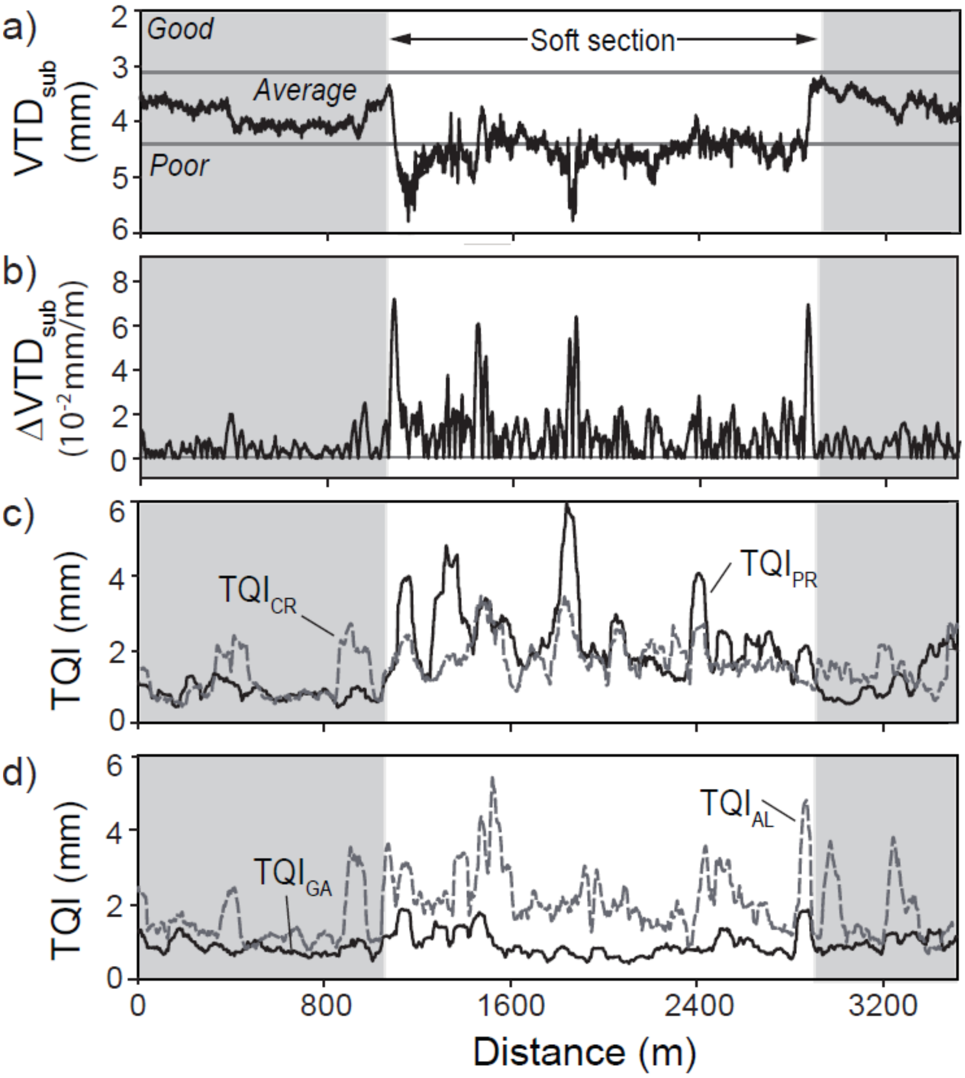

Figure 4-9. Plots of the (a) VTD_{sub}, (b) ΔVTD_{sub}, (c) TQI_{PR} and TQI_{CR}, and (d) TQI_{GA} and TQI_{AL} for a 2 km example of *poor* track conditions.

Plots of TQI_{CR} versus VTD_{sub} are presented in **Figure 4-11**a and TQI_{CR} versus ΔVTD_{sub} in **Figure 4-11**b. The plots in **Figure 4-11** show that at any given VTD_{sub} or ΔVTD_{sub} there is a spectrum of TQI_{CR} values which results in a poor correlation with coefficient of determination (R^2) of 0.38 and 0.40 from the linear regression. Plots for TQI_{PR}, TQI_{GA}, and TQI_{AL} show very similar plots with R^2 values ranging from 0.35 to 0.56. As these regressions are developed from 2,235,328 data points, there is a > 99.9% probability that an underlying correlation between increasing TQI, VTD_{sub} and ΔVTD_{sub} exists (Smith, 2015). Non-linear correlations and further data processing to match the peaks of TQI and ΔVTD_{sub} that are offset from one another (**Figure 4-9**) to account for the accuracy of the GPS locations, and selection of TQI from differing track geometry measurements did not result in a stronger relationship between TQI, VTD_{sub} and ΔVTD_{sub}. This relationship is obscured by the impact of maintenance. The authors have not included the equations so that they are not used to predict track conditions.

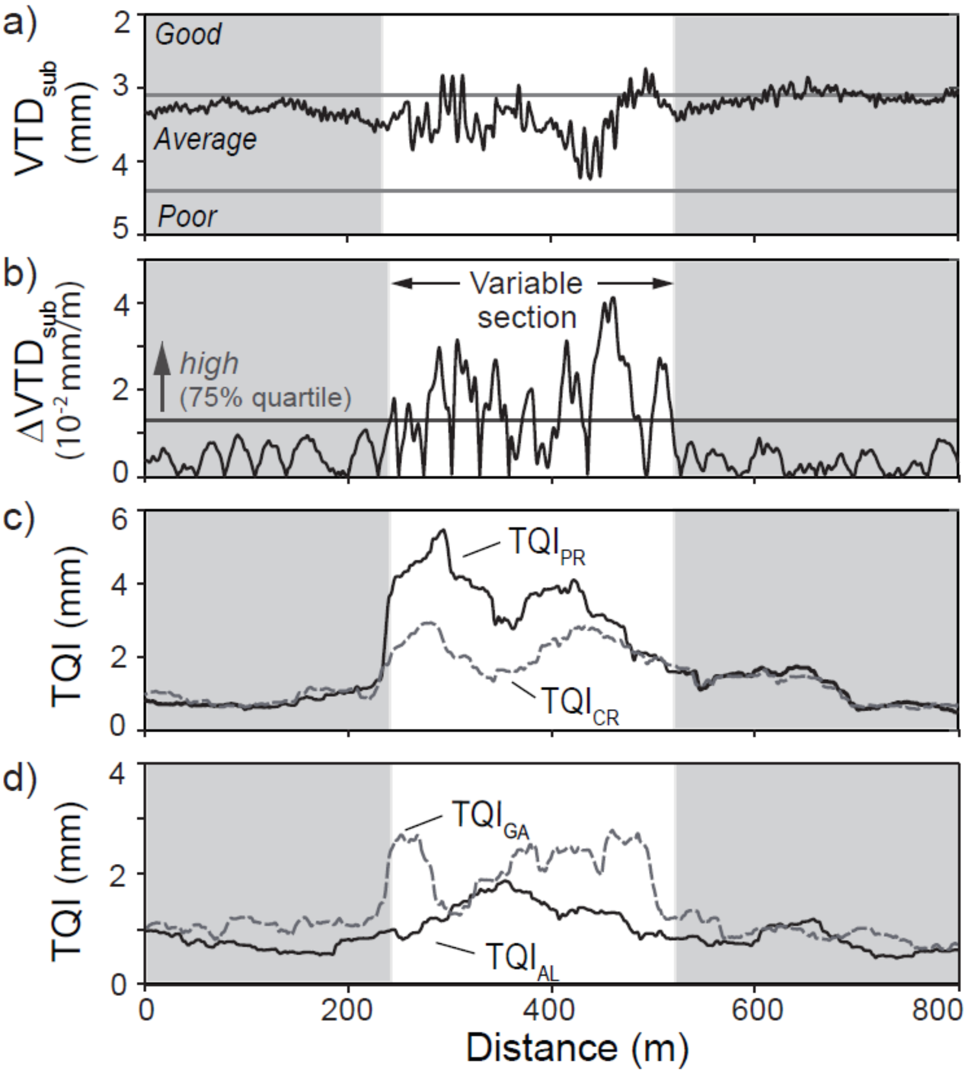

Figure 4-10. Plots of the (a) VTD_{sub}, (b) ΔVTD_{sub}, (c) TQI_{PR} and TQI_{CR}, and (d) TQI_{GA} and TQI_{AL} for an 800 m section of track of *average* to *good* track conditions (based on VTD_{sub}, with a 270 m section of track with elevated ΔVTD_{sub}.

A simpler comparison between TQI, VTD_{sub} and ΔVTD_{sub} can be shown with the distribution of TQI values within the differing catagories of VTD_{sub} and ΔVTD_{sub} (**Figure 4-12**). **Figure 4-12**a presents the skewed, but similar, distributions of TQI_{PR} values from both the Prairies and Canadian Shield subdivisions from a single measurement of track geometry; similar plots of the distributions of TQI_{CR}, TQI_{GA}, and TQI_{AL} are presented in **Figure 4-13**. The range of values for the TQI values range between 0.5 and 5.0 mm, with the exception of TQI_{AL} that has a narrower range of 0.5 and 3.0 mm. **Figure 4-12**b presents the distributions of the TQI_{PR} for track divided into subsets of good and poor VTD_{sub}; and, **Figure 4-12**c presents the distributions of the TQI_{PR} for track divided into subsets of high and low ΔVTD_{sub}.

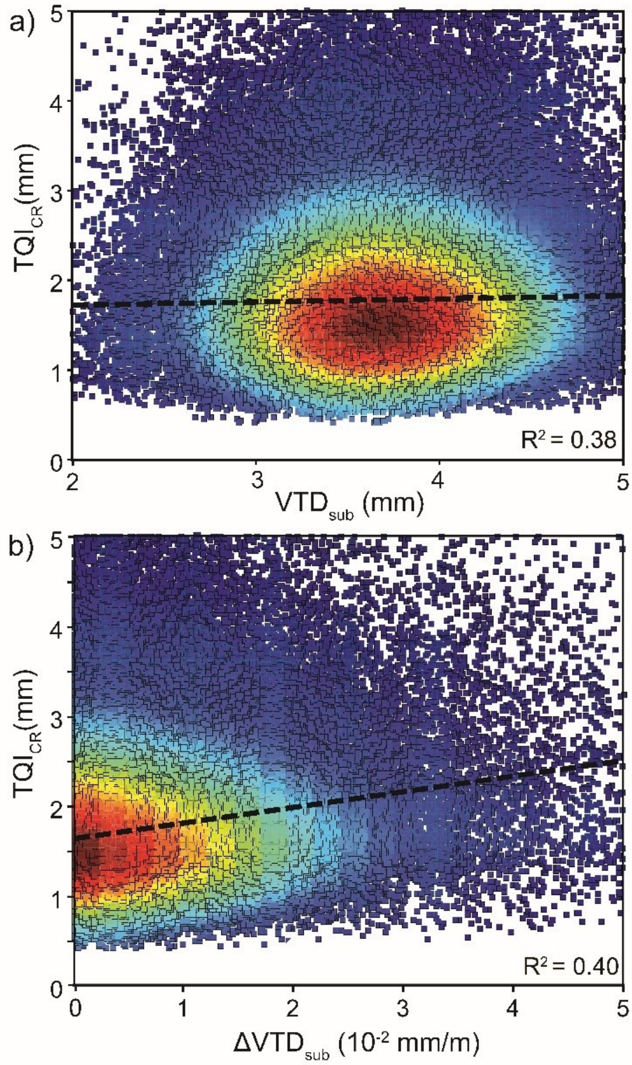

Figure 4-11. Plots of TQI_{CR} versus (a) VTD_{sub} and (b) ΔVTD_{sub}. The color denotes the density of the data points, with the highest density shown in red; the colors intended to be a qualitative representation and thus a legend is not provided. The dashed lines are the linear fits to the data presented for discussion.

It is clear from the distributions presented in **Figure 4-12**b-c that the trend of higher VTD metrics resulting higher TQI_{PR} values does exist within the data. There is a significant amount of overlap in the distributions of TQI_{PR} for the different catagories of VTD_{sub} and ΔVTD_{sub}, this was also evident in this data as plotted in **Figure 4-9**, and an again shows the inability to predict TQI_{PR} from VTD_{sub} and ΔVTD_{sub}. The poor and high categories show a wider distribution (**Figure 4-12**b-c), where the good and low categories show a more concentrated distribution. From **Figure 4-12**b-c, the categories of VTD_{sub} are a slightly better discriminator of expected TQI_{PR} than the categories of ΔVTD_{sub} as the difference between the peaks (modes) in **Figure 4-12** is greater than those in **Figure 4-12**c. However, the categories of ΔVTD_{sub} appear to be better indicators of high TQI_{PR} as they show very similar densities for the differing categories of VTD_{sub} for TQI_{PR} > 4 mm (**Figure 4-12**b), in contrast to the large difference between the densities between the differing ΔVTD_{sub} categories (**Figure 4-12**c). Similar observations can be made for the distributions of TQI_{CR}, TQI_{GA}, and TQI_{AL}, which are presented in **Figure 4-14**. The exceptions are the relationship between the alignment and crosslevel with the VTD_{sub}, for which the distributions for good and poor VTD_{sub} were nearly identical, suggesting that VTD_{sub} does not impact the roughness of the alignment and crosslevel. The contrast between the strength of the relationship between defects and VTD but not between TQI and VTD led to further examination of the TQI distributions to determine why this difference exists. The authors suggest that this difference is a result of the use of threshold values for geometry in the evaluation of defects, versus analyses of the full spectrum of possible TQI values. And, that the poor VTD_{sub} and high ΔVTD_{sub} have a much higher representation at higher values of TQI (**Figure 4-12**). Thus, an arbitrary threshold of 3.0 mm was applied to the TQI and the locations that exceeded this value were given the same treatment of as the geometry defects. The sections of track from both subdivisions were divided into 16 categories based on VTD_{sub} and ΔVTD_{sub}. **Figure 4-15**a and **Figure 4-15**b shows the percentage of total amount of track with TQI_{GA} and surface related TQI (TQI_{PR} and TQI_{CR}) that exceed the 3.0 mm threshold. The correlation between TQI_{GA} and surface related TQI was strong with ΔVTD_{sub} and showed no correlation with VTD_{sub}. This shows the significance of ΔVTD_{sub} in development of poor track roughness, with the average percentage of track with TQI_{GA} > 3.0 mm increase from 9.4% to 19.6% when ΔVTD_{sub} changes from low (< 25%) to high (> 75%). This impact is even more pronounced for the surface related TQIs where change in ΔVTD_{sub} from low to high increase the average percent of track with TQI > 3.0 mm from 21.0 to 47.2%.

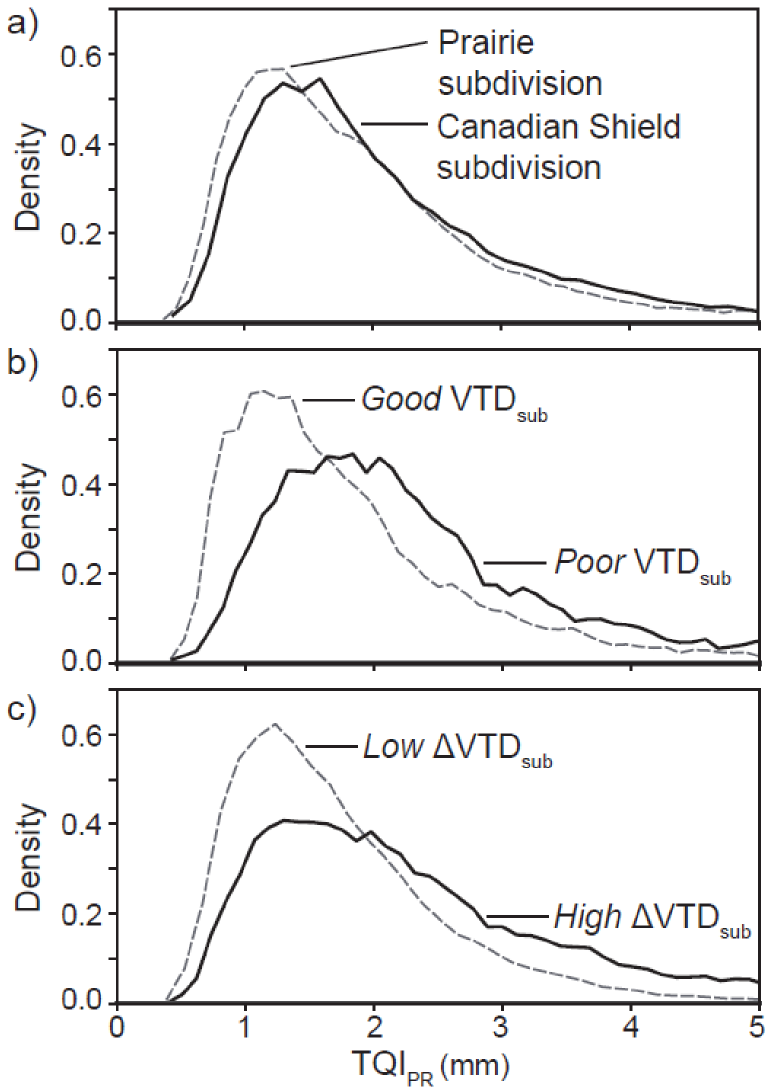

Figure 4-12. Plots comparing the distribution of (a) the TQI_{PR} for the Prairie and the Canadian Shield subdivision, (b) the TQI_{PR} for track divided into subsets of *good* and *poor* VTD_{sub}, and (c) the TQI_{PR} for track divided into subsets of *high* and *low* ΔVTD_{sub}. Where, *good* VTD_{sub} is < 3.1 mm, *poor* VTD_{sub} is > 4.4 mm, *low* ΔVTD_{sub} is < 0.003 mm/m, and *high* ΔVTD_{sub} is > 0.013 mm/m.

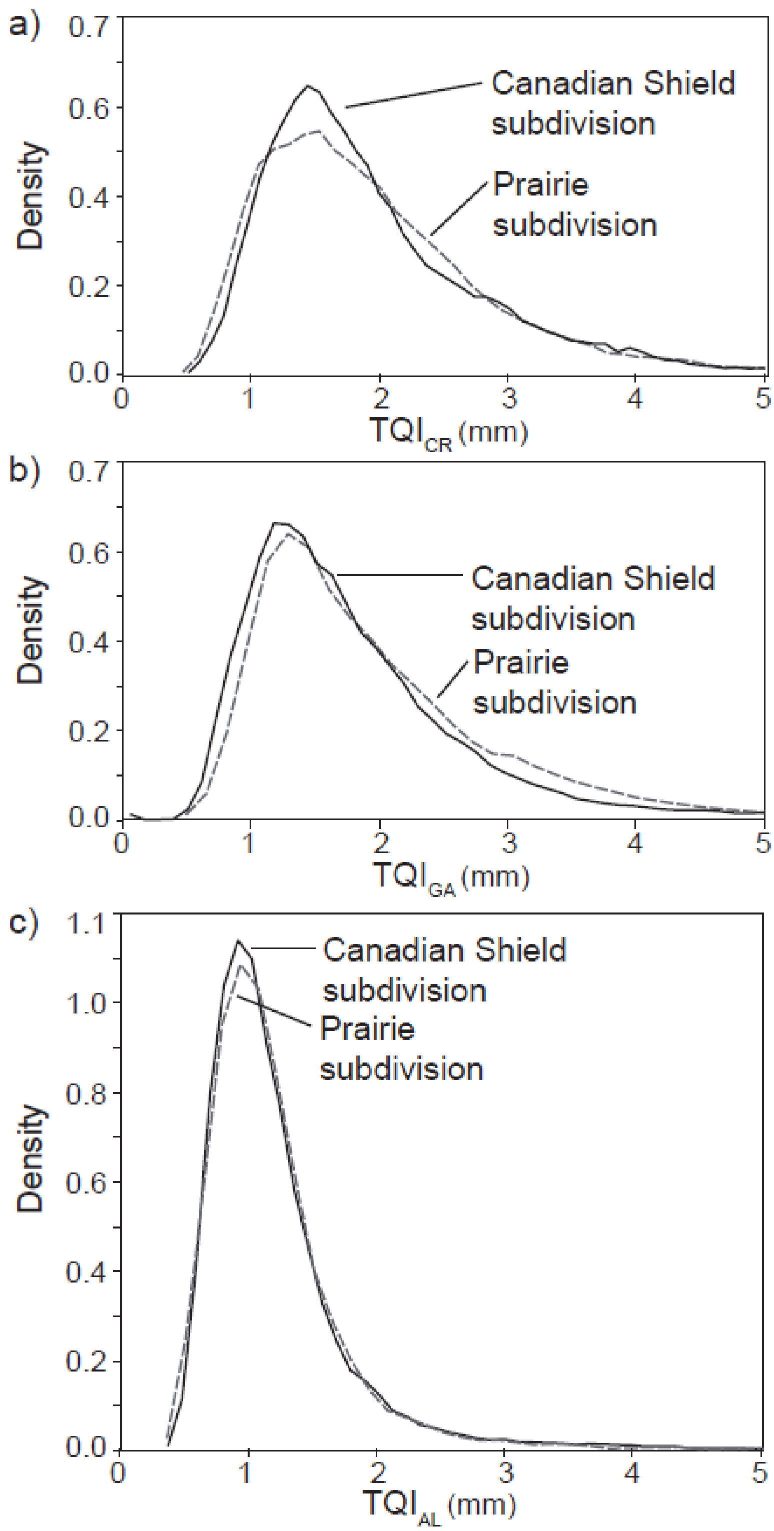

Figure 4-13. Plots of the distribution of (a) TQI_{CR}, (b) TQI_{GA}, and (c) TQI_{AL} for both the Prairie and Canadian Shield subdivisions.

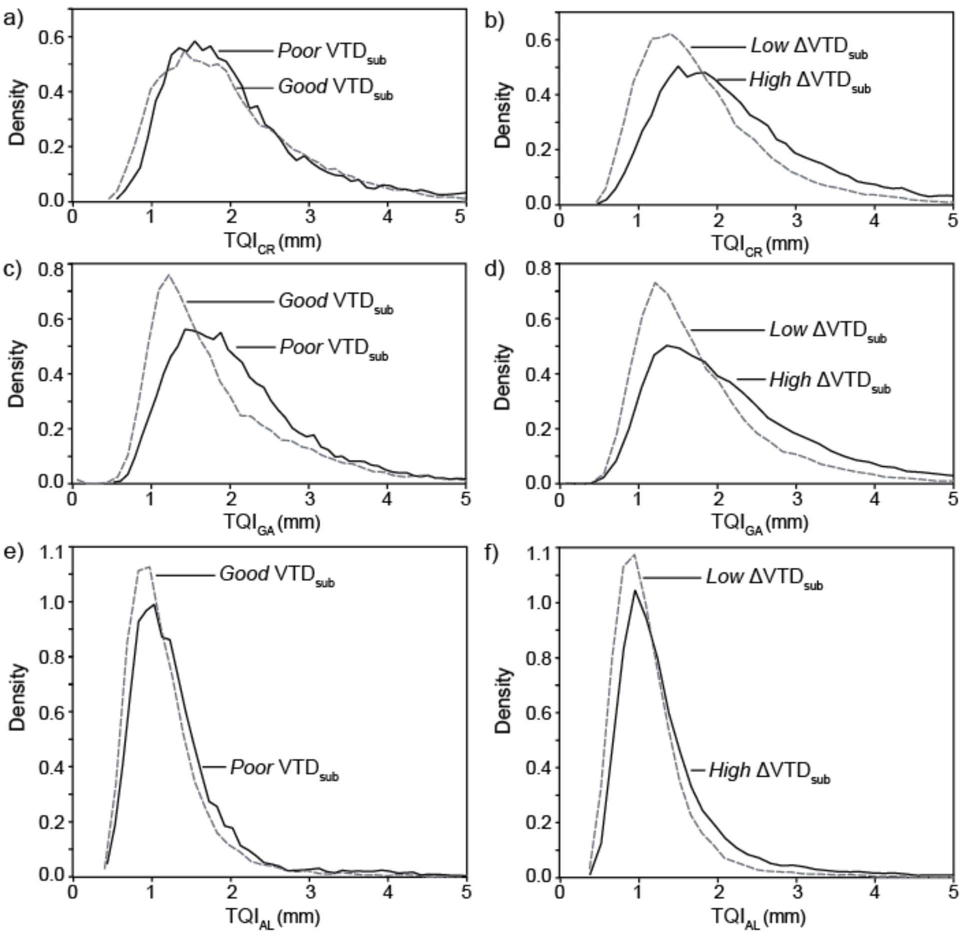

Figure 4-14. Plots of the distribution of: TQI_{CR} divided in into subsets of (a) *good* and *poor* VTD_{sub} and (b) *high* and *low* ΔVTD_{sub}; TQI_{GA} divided in into subsets of (c) *good* and *poor* VTD_{sub} and (d) *high* and *low* ΔVTD_{sub}; and, TQI_{AL} divided in into subsets of (e) *good* and *poor* VTD_{sub} and (f) *high* and *low* ΔVTD_{sub}. Where, *good* VTD_{sub} is < 3.1 mm, *poor* VTD_{sub} is > 4.4 mm, *low* ΔVTD_{sub} is < 0.003 mm/m, and *high* ΔVTD_{sub} is > 0.013 mm/m.

The surface related TQIs (**Figure 4-15**b) could be further divided into TQI_{PR} (**Figure 4-16**a), TQI_{CR} (**Figure 4-16**b). TQI_{CR} shows a strong correlation with ΔVTD_{sub} whereas for TQI_{PR} both VTD_{sub} and ΔVTD_{sub} are contributing. A sensitivity analysis was also conducted on the threshold value for TQI. It was found that these trends become obscure if the threshold is reduced below 2.5 mm, and remains strong with increasing thresholds until the dataset above the threshold becomes too small to trend.

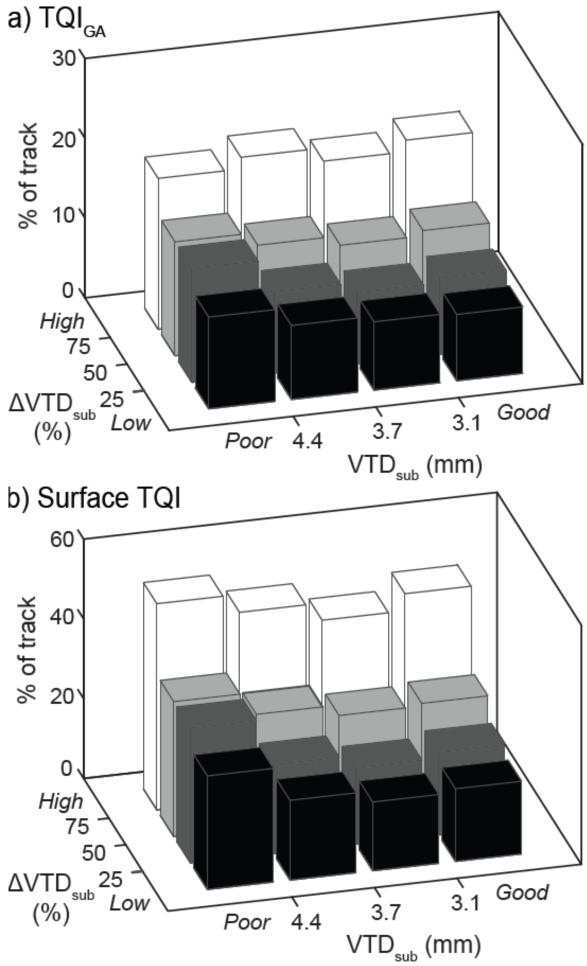

Figure 4-15. Plots of the percent of total length of track with (a) $TQI_{GA} > 3$ mm and (b) surface related TQI > 3 mm within the divisions of VTD_{sub} and ΔVTD_{sub}. These plots include data from both the subdivision in the Prairies and the subdivision in the Canadian Shield.

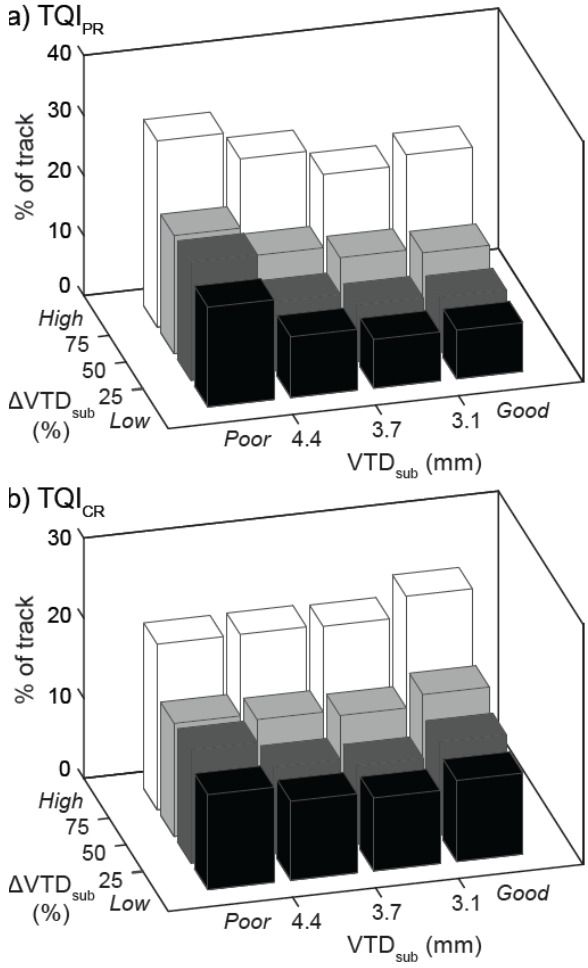

Figure 4-16. Plots of the percent of total length of track with (a) TQI_{GA} > 3 mm and (b) surface related TQI > 3 mm within the divisions of VTD_{sub} and ΔVTD_{sub}. These plots include data from both the subdivision in the Prairies and the subdivision in the Canadian Shield.

4.7. Conclusions

More than 800 km of VTD and track geometry measurements from two subdivisions from different physiographic region of Canada were processed to evaluate the impact of substructure condition on track performance. The substructure condition was quantified by the both VTD and track geometry measurements. The VTD measurements were filtered for a VTD_{sub} that is representative of the subgrade conditions and was put into context with AREMA suggested track modulus values. The slope of the VTD_{sub} versus distance, ΔVTD_{sub}, was used to quantify changes in track stiffness as they appear within VTD measurements. The geometry measurement data consisted of three years, 15 measurements, of priority and urgent defects and track quality indices from the two subdivisions.

From the analysis of VTD_{sub} and ΔVTD_{sub}, and the comparison to generation of priority and urgent defects and TQI, the following conclusions were developed. The geometry of the track, represented as the geometry measurements or as TQI does have an underlying relationship with the subgrade conditions, more so with changes in VTD (ΔVTD_{sub}), and thus variation of modulus, than the magnitude of VTD_{sub}. This relationship is obscured in the data due to the effects of maintenance that is regularly carried out to minimize the development of poor track conditions. The use of threshold values for both the track geometry measurements to obtain defects, and for the TQI shows a strong correlation with VTD_{sub} and $\Delta VTD_{\mathrm{sub}}$. These results show that the locations that have low modulus (higher VTD) and a high variability in the modulus are those that are difficult to maintain, and at which maintenance is not always able to keep up with the degradation of the track geometry. These VTD measurements evaluate the underlying causes that result in the degradation of track conditions and allow for the identification of sections where it most likely that maintenance will not always be able to keep up with degradation; even if maintenance has done so recently.

The two subdivisions showed similar distributions of VTD_{sub}, ΔVTD_{sub} TQI, and defects per km, despite the being in different physiographic regions. This similarity provides some confidence that these results and relationships are more widely applicable.

A different approach for combining the track geometry and track deflection measurement is presented in Appendix B along with an example of how to implement these types of data into the maintenance planning.

Note: all of the threshold values, with the exception of those in regulations, should be optimized for the specific conditions and goals of a railway operator and the class of track before adopting them for the assessment of track.

4.8. Acknowledgement

The authors would like to thank the Canadian National Railway for their support and facilitation of this project. This research was made possible through the Canadian Rail Research Laboratory (CaRRL) (www.carrl.ca), which is funded by the Natural Sciences and Engineering Research Council of Canada (NSERC), Canadian Pacific Railway, Canadian National Railway, the Association of American Railways – Transportation Technology Centre Inc., the National Research Council of Canada, Transport Canada and Alberta Innovates – Technology Futures.

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5. Chapter 5: Quantifying the effectiveness of methods used to improve railway track performance over soft subgrades: methodology and case study

5.1. Contribution of the Ph.D. candidate

All the work presented in this chapter, includingdata processing, review of literature, analysis and discussion of the results and writing of the text is carried out by the Ph.D. candidate. As supervisor, Dr. M.T. Hendry has reviewed all parts of the work. This chapter is submitted to the journal of Transportation Engineering with the following citation:

Roghani, A., Macciotta, R., and Hendry, M.T. (2016). "Quantifying the effectiveness of methods used to improve railway track performance over soft subgrades: methodology and case study" Journal of Transportation Engineering, American Society of Civil Engineering (ASCE).

5.2. Abstract

This paper presents a methodology for quantifying the effectiveness of different methods used to improve the railway track performance on soft subgrades. This methodology consists of quantifying the changes in track stiffness from Vertical Track Deflection (VTD) measurements taken before and after the track was upgraded, and the evaluation of the roughness of the track that has developed since the track was upgraded. A project was presented within this paper as a case study to explain the steps of this methodology. These upgrades consisted of changing the rail from 49.6 kg/m (100 lb/yd) bolted rail to 57 kg/m (115 lb/yd) continuously welded rail; embankment reconstruction, and using a layer of geogrid at the subballast - subgrade interface.

The results of the study showed that the VTD measurements were capable of measuring changes in track deflection, and thus modulus, due to the upgrading of the track structures with a high enough resolution to distinguish between the differing test sections. The track geometry measurements suggested that the not enough time or train traffic had passed to degrade the track

geometry to a level that would start indicating issues in performance. The relative effectiveness of the different remediation methods at this study site were also evaluated. This showed that the replacement of jointed rail with heavier CWR significantly increased the track stiffness, more than the excavation of the subgrade and reconstruction of the embankment. The combined effect of CWR and the substructure upgrades further improved the track modulus. And, that the geogrid could be used with CWR to reduce the amount of subballast required without an increase in track deflection.

5.3. Introduction

Soft subgrades have been associated with large track deflections, higher rail bending stresses, increased wear of the track components, and the deterioration of track geometry (Ebersöhn *et al.*, 1993; Cai *et al.*, 1994; Read *et al.*, 1994; Selig and Li, 1994; Selig & Waters, 1994; Esveld, 2001; Sussmann *et al.*, 2001; Zarembaski & Palese, 2003; Davis *et al.*, 2003; Hendry *et al.*, 2008; Li *et al.*, 2015). Railway operators have tried a wide variety of methods to improve the performance of track where poor subgrades exist. This typically includes increasing the weight of rail, installing longer ties, or increasing the depth of ballast so to further distribute stress and thus displacement (Roghani *et al*., 2015). More invasive remediation is sometimes required; this can be in the form of piles, injection or mixing of cement into the subgrade, or the excavation and reconstruction of embankments and replacement of subgrade materials (Hebib *et al*., 2003; Hendry *et al*., 2011;). These efforts to improve track are conducted at great expense, but the improvement achieved in the track performance has not been readily quantifiable. A site is considered to have been successfully remediated if the problems no longer occur. Success may vary from the problem no longer existing to it becoming manageable with extensive and continuous maintenance. Without the ability to quantify the effectiveness of the remediation there is no means to compare alternative methods or to determine if a solution was underbuilt or overbuilt. Thus, it has been difficult to assess the value obtained from the investment in improving track and substructures.

This paper presents a case study of a section of track that has undergone extensive upgrading. This work was designed as an experiment to compare differing remediation methods at a single site. This case study is presented to both show a methodology for quantifying the improvement in track performance using vertical track deflection (VTD) measurements and track geometry measurements; and, to present the results of this experiment of differing reconstruction methods for improving track over soft subgrade conditions.

5.4. Description of the study site and experiment

The study site is located on the Pierre to Rapid City subdivision (PRCS), a 272 km track that connects the cities of Pierre and Rapid City, South Dakota. This subdivision is operated by the Dakota, Minnesota and Eastern Railroad (DM&E). When the track upgrades were undertaken in 2012, the DM&E was a subsidiary of Canadian Pacific (CP), which acquired the DM&E in 2007/08 with the intent to extend the railroad into the Powder River Basin and the nearby coalmines. The DM&E was purchased by short line operator Genesee and Wyoming Inc. in 2014. The PRCS has been classified as 'excepted track', which is the lowest FRA classification, and carries a 16 km/h (10 mph) speed limit for freight trains; revenue passenger trains cannot use the track. The performance of the PRCS has been limited by challenging local subgrade conditions including deposits from the Bad River as well as the Pierre Shales (Searight, 1952). The subgrade within the river valley is comprised of highly plastic clay that was washed into these valleys and deposited as alluvium. The stiffness of the subgrades within this valley is variable, stiffest near the valley wall and softest at the bottom of the valley (Roghani & Hendry, 2016a)

The experimental site is located within the Bad River valley between mileposts 503.6 and 504, approximately 30 km southwest of the city of Pierre (**Figure 5-1**). Before the upgrades, the track superstructure along the test sections consisted of 49.6 kg/m (100 lb/m) bolted rail and wooden ties in relatively poor conditions, with many loose or missing spikes (**Figure 5-2a**). Five exploratory trenches were excavated into the embankment before the start of construction.

Photographs from these excavations are presented in **Figure 5-2b** and **Figure 5-2c**. Samples of subgrade materials showed high plastic clays with water contents between 34 to 42%, plastic limit from 22 to 24%, and liquid limits from 80 to 100%. Testing also showed that the ballast and subballast were heavily fouled with these clays. It was observed during the excavations that the subgrade has undergone significant plastic deformation, resulting in formation of water filled ballast pocket beneath the track and a heave near the shoulder (**Figure 5-2b** and **Figure 5-2c**).

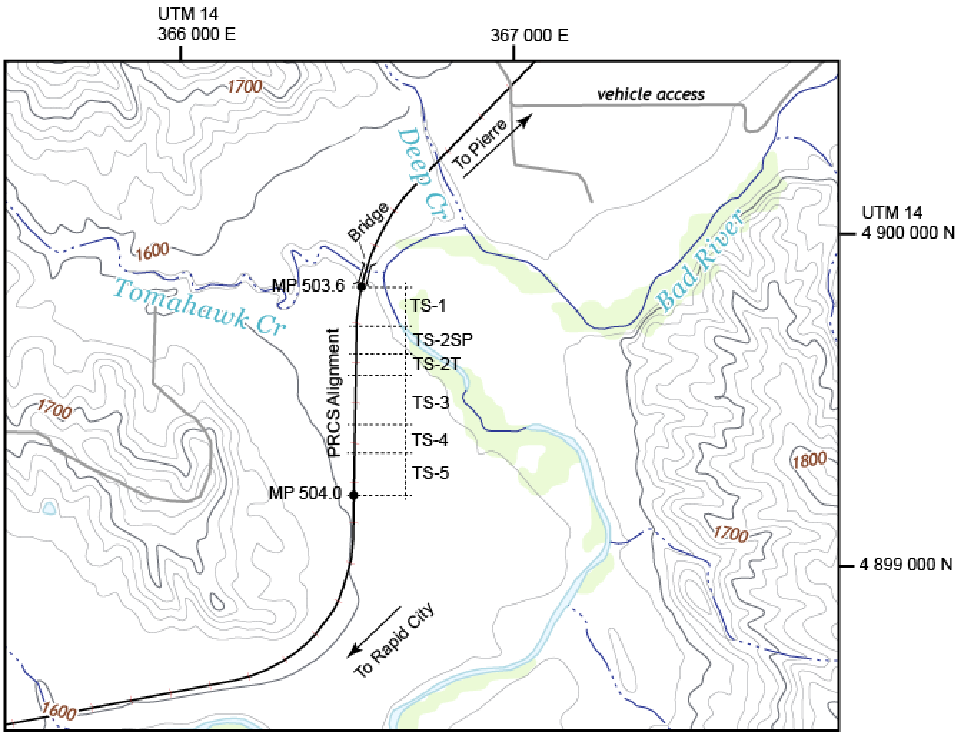

Figure 5-1. The map shows the study site and the surrounding terrain. The location of differing test sections are indicated (USGS National Geospatial Technical Operations Center, 2015).

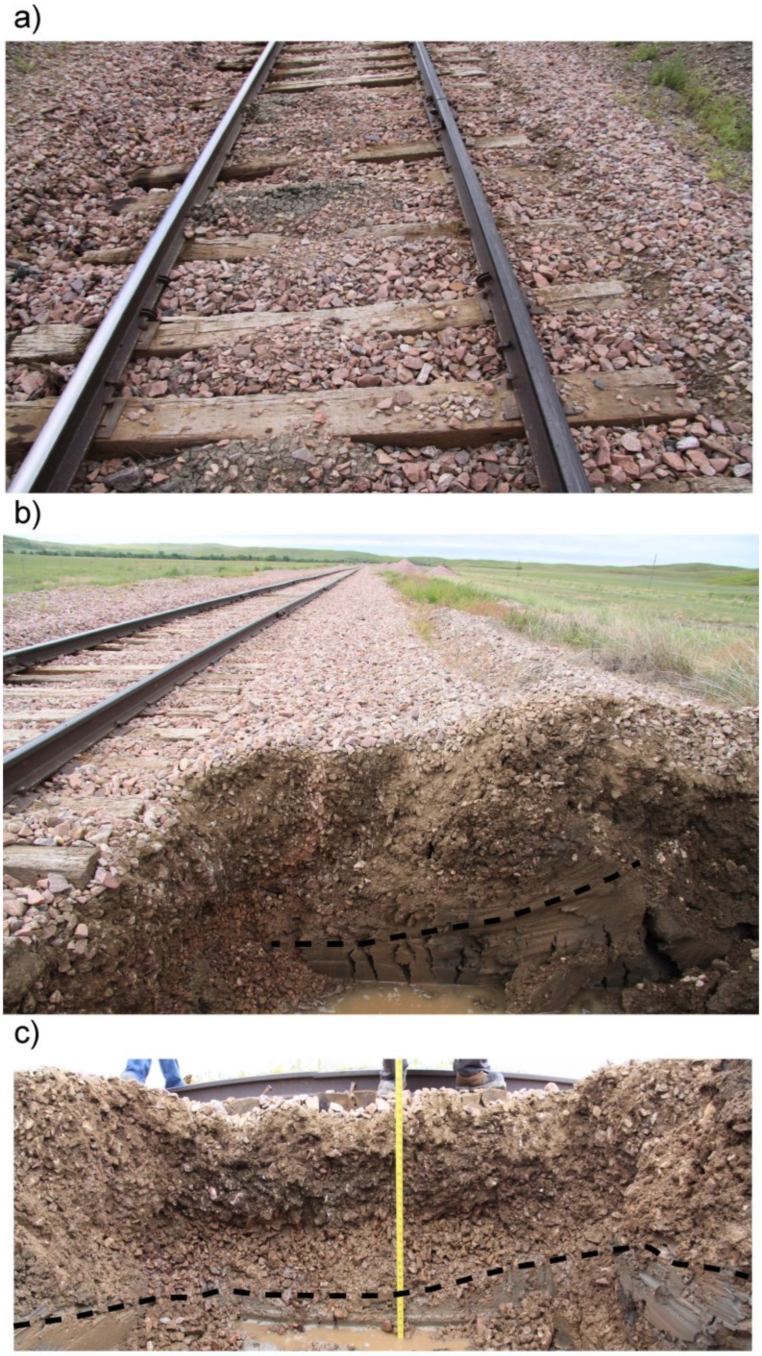

Figure 5-2. Photographs of (a) the relatively poor condition of the track before the upgrading and, (b and c) the construction of the embankment and the subgrade as observed in test pits. The interface between embankment material and subgrade is shown with dashed lines.

5.5. Experimental track sections

The excavation and reconstruction of the site occurred between July 9 and 26, 2012. This was designed as an experiment with test sections (TS-1 to TS-5) of differing methods for improving the performance of the track (**Figure 5-1** and **Figure 5-3**). The details of the construction of each test section are presented in **Table 5-1**, the location of the sections are shown in **Figure 5-1**, and final compositions of the structures is shown in **Figure 5-3**. The experiment was designed to evaluate the impact of reconstructing the track embankment to AREMA suggested thicknesses of materials (TS-1); the replacement of the rail with a heavier CWR (TS-3, TS-4 and TS-5); and, to evaluate whether a geogrid can be used to replace 150 mm of subballast as suggested by the manufacturer (TS-2 and TS-3). For test sections TS-1 through TS-4, the embankment and subgrade were excavated down to firmer clay. The depth of excavation varied between 0.3 and 1.0 m among the test sections, with the minimum excavation at the norther portion of TS-2. This last corresponds to spiral (curved) track. Local material was used to fill the excavation to the surface of the surrounding terrain, and the embankment constructed on top. The design specified that this fill was to be compacted to 90% of the Modified Standard Proctor test (ASTM D1557). Vane Shear tests (VST) were conducted on the compacted fill. The VST showed that the compacted fill had a strength between 150 kPa and 280 kPa where it was well compacted (Benesch, 2012). The VST value in the portion of TS-2 beneath the curve showed an undrained shear strength of only 64 kPa, and this was interpreted as being the result of poor compaction.

Table 5-1. The details of construction of the test sections installed between mileposts 503.6 and 504 on the Pierre to Rapid City Subdivision.

Section No.	Composition of embankment	Superstructure	Track alignment
TS-1	300 mm subballast, 300 mm new ballast,	Original 49.6 kg/m bolted rail	Curve
TS-2[1]	Geogrid[2] on top of the compacted fill, 150 mm subballast, 300 mm new ballast	Original 49.6 kg/m bolted rail	Spiral and Tangent
TS-3	Geogrid on top of the compacted fill, 150 mm subballast, 300 mm new ballast	Track upgraded to 57.0 kg/m CWR	Tangent
TS-4	300 mm subballast, 300 mm new ballast	Track upgraded to 57.0 kg/m CWR	Tangent
TS-5	Variable original conditions (not upgraded)	Track upgraded to 57.0 kg/m CWR	Tangent

[1] This section is further divided into *TS-2SP* which has a spiral and *TS-2T* which has tangent track.

[2] Tensar TriAx TX 160 (Tensar, 2012).

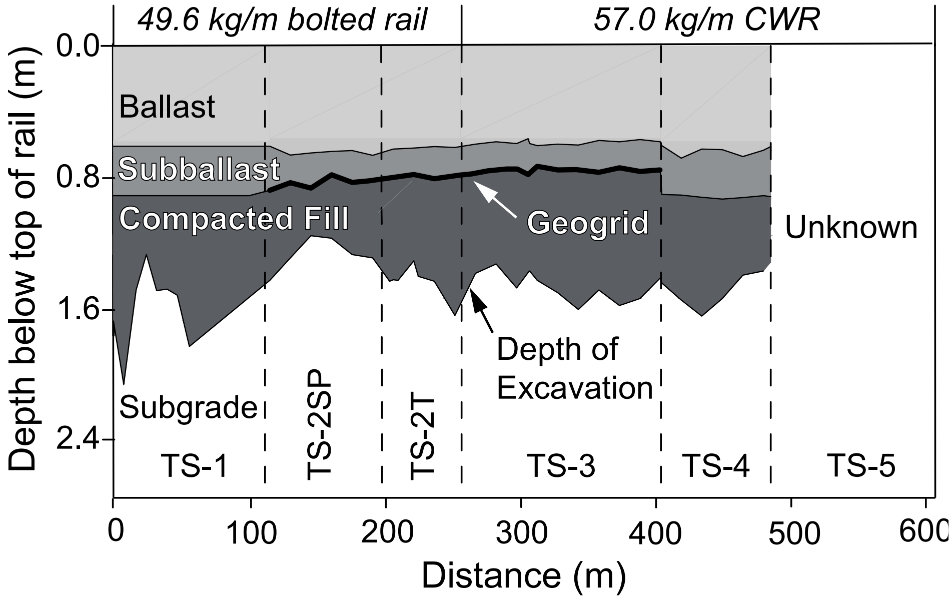

Figure 5-3. Section through the centerline of the embankment showing the depth of excavation, and the materials used for the reconstruction of the embankment and track within each of the test sections based on Benesch (2012).

5.6. Measurement systems

The data used in this analysis consisted of two sets of VTD measurements recorded before and after the upgrade, and one set of track geometry measurements recorded after the upgrade. This section provides a description of these measurements and how they are used as measurements of performance.

5.6.1. Vertical Track Deflection measurements

The MRail rolling deflection measurement system (**Figure 5-4**), developed at University of Nebraska in collaboration with Federal Railroad Administration (Norman, 2004; Norman *et al.*, 2004; McVey *et al.*, 2005; Farritor, 2006; McVey, 2006; Arnold *et al.*, 2006, Lu, 2008; Greisen, 2010; Farritor & Fateh, 2013), was used to measure VTD over the PRCS on April 17, 2012 (100 days before upgrades) and on July 17, 2014 (721 days after the upgrades). These data sets were filtered to remove wavelengths of less than 20 m to obtain VTD_{sub} which is a measure of the stiffness of the subgrade and embankment construction (Roghani & Hendry, 2016a). Portions of this data has been previously presented in Roghani & Hendry (2016a), to show the ability of the system to map the location of soft subgrades. This data was also used to calculate the change in VTD$_{sub}$, ΔVTD$_{sub}$, which corresponds to the change in track stiffness. The ΔVTD$_{sub}$ is calculated as the absolute value of secant slope of the VTD$_{sub}$ over 20m length of track (Roghani & Hendry, 2016b) (Equation 5-1).

$$\Delta \mathrm{VTD}_{sub}\ (x) = \ |\mathrm{VTD}_{sub}\ (\ x + 10) - \ \mathrm{VTD}_{sub}\ (\ x - 10)|/20\ , \qquad \text{Equation 5-1}$$

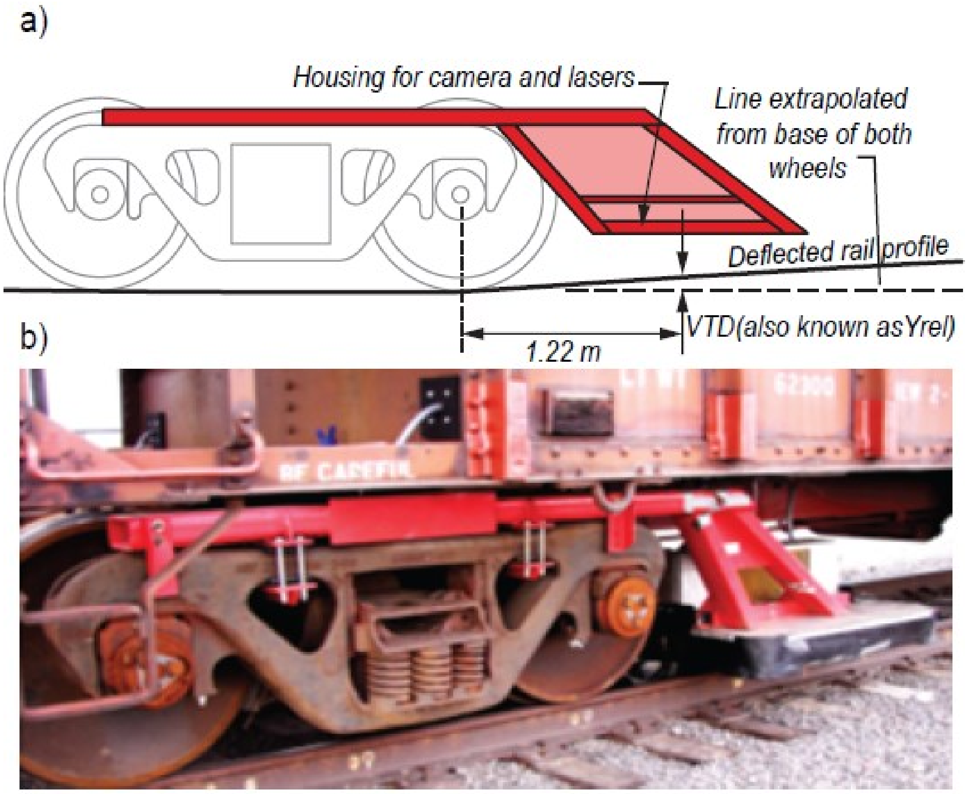

Figure 5-4. (a) The configuration of the MRail system and definition of VTD measurements (*after* Roghani & Hendry, 2016a) and (b) a photograph of the MRail system installed on a car to collect the VTD measurements for this study.

The same rail car, axle loads (29.9 ± 1.5 tonnes) and instrumentation, were used for both measurement runs. The load from the car adjacent to measurement system varied between measurement runs, and this resulted in the different VTD$_{sub}$ values that can be observed in **Figure 5-5** (Roghani & Hendry, 2016a). **Figure 5-5** compares the VTD$_{sub}$ values for 2012 and 2014 measurement over a 48 km length of track, and shows that the measurements from 2014 are notably higher with the exception of the test sections.

The ability to maintain operable track geometry is the primary function of the structure beneath the track. Roghani & Hendry (2016b) showed that the ability of the track structures to maintain its geometry is strongly a result of its stiffness, quantified with VTD_{sub}, and the variation in stiffness, quantified with the ΔVTD_{sub}. Thus, these VTD parameters are used to quantify the stiffness of track and also to compare the ability of test sections to maintain track geometry under traffic.

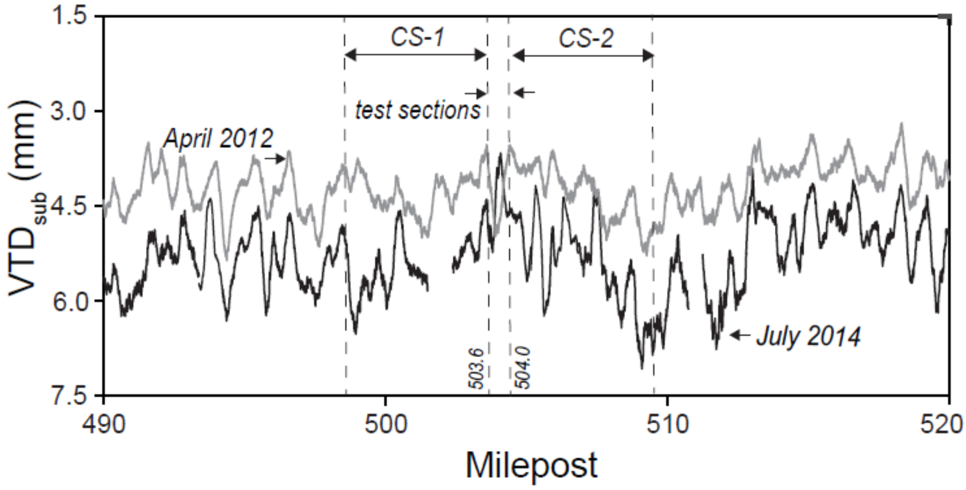

Figure 5-5. Plot comparing VTD_{sub} over a 48-km section from 2012 and 2014 measurements on the PRCS and the location of control and test sections.

5.6.2. Track geometry measurements

The track geometry was measured over the PRCS on October 10th 2013; 541 days after the completion of the upgrades. The geometry measurements included alignment, profile, gauge, and crosslevel. These geometry measurements are standardized and regulated (AREMA, 2012; FRA, 2007). Alignment is the horizontal deviation of the gauge side (inside) of the rail from a line subtended from two points 18.9 m (62 ft) apart on the surface measured at the midpoint of that line (mid-chord offset). Profile is the mid-chord offset measured vertically on the surface of the rail. Gauge is the distance between two rails measured 16 mm (5/8 inch) below the top of the rail with standard gauge equal to the 1435.1 mm (56.5 inches). Crosslevel (super-elevation at curves) is the elevation difference between both rails on a tangent or curved track (AREMA, 2012; FRA, 2007).

Track quality indices (TQI) are common metrics used to quantify the track geometry measurements and assess the performance of track. TQI is a statistical measure such as maximum, standard deviation, or average absolute error, that aggregates track geometry measurements over a segment of track (Hyslip, 2002; El-Sibaie & Zhang, 2004; FRA, 2004, 2005; Berawi *et al.*, 2010; Sadeghi & Askarinejad, 2010). Zarembaski & Bonaventura (2010) suggest that the magnitude of TQI should be kept to a minimum to reduce the dynamic forces and thus reduce the rate of deterioration of track components and rolling stock. TQI is typically used to prioritize track maintenance, support decision for track re-classification, quantify the rate of track geometry degradation, and assess the quality and effectiveness of track maintenance (AREMA, 2012).

The standard deviation (Equation 5-2) of geometric measures evaluated for a section of track provides a simple TQI that is a representative of track roughness as more complex formulation for TQI (ORE, 1981). A higher TQI (standard deviation) suggests the track to be rougher and in poorer condition.

$$TQI = \sqrt{\frac{\sum_{i=1}^{N}(x_i - \bar{x})^2}{N}}, \qquad \text{Equation 5-2}$$

Where, x_i is the deviation of geometry parameter measured at point i, N is the number of sequential measurement datum and $\bar{x}$ is the average of the data within the test section.

The track over the study site was tamped after it was reconstructed and no further maintenance was carried out before the track geometry was measured on October 10th 2013 (541 days after reconstruction). Thus, the track roughness, represented by the TQI, developed since the end of construction and higher TQI values are interpreted to represent a poorer ability of the different track structures to maintain track geometry.

5.7. Presentation of measured data

The measured VTD from April 2012 and July 2014 are presented in **Figure 5-6a** and **Figure 5-6b**, respectively. The 2012 data shows high variability over the length of the test sections, with a reduction of variability in 2014 in the test sections where CWR was installed (TS- 3, TS-4, and TS-5). This is similar to the results presented for the differences between jointed rail and CWR in Roghani & Hendry (2016a). Following Roghani & Hendry (2016a), the VTD measurements along the test section were filtered with a 20-m moving average for measurements before and after the upgrades (**Figure 5-6c**). The results presented in **Figure 5-6c** have not been adjusted for the differing axle loads of the adjacent rail car. It is apparent from a comparison of the 2012 and 2014 VTD_{sub} measurements that a softer section existed through the study site and section were improved with reconstructed. There is also a notably elevated VTD_{sub} and ΔVTD_{sub} through TS-2SP where there was the least amount of excavation (**Figure 5-3**) and where fill material is thought to be under-compacted.

The track geometry measurements from the study site are presented in **Figure 5-7**. The track geometry for this section is in good condition; there are no locations at which the geometry exceeds the thresholds defined by the FRA for either this class of track, or even the much more stringent thresholds of a Class 5 track (FRA, 2007). The standard deviation of the geometry measurements was used to calculate the TQI for the track profile (TQI_{PR}), gauge (TQI_{GA}), crosslevel (TQI_{CR}), and alignment (TQI_{AL}) at each test section and the results are presented in **Figure 5-8** TS-2 was subdivided into two sub-sections of spiral and tangent (straight) alignment (TS-2SP and TS-2T, respectively) and their TQIs are calculated separately as recommended in AREMA (2012). Two control sections, CS-1 and CS-2, were also included in this analysis to allow for the comparison of track which was neither tamped nor upgraded. CS-1 starts from the end of TS-1 and extends 8 km towards Pierre, and CS-2 starts from the end of TS-5 and extends 8 km towards Rapid City (**Figure 5-5**).

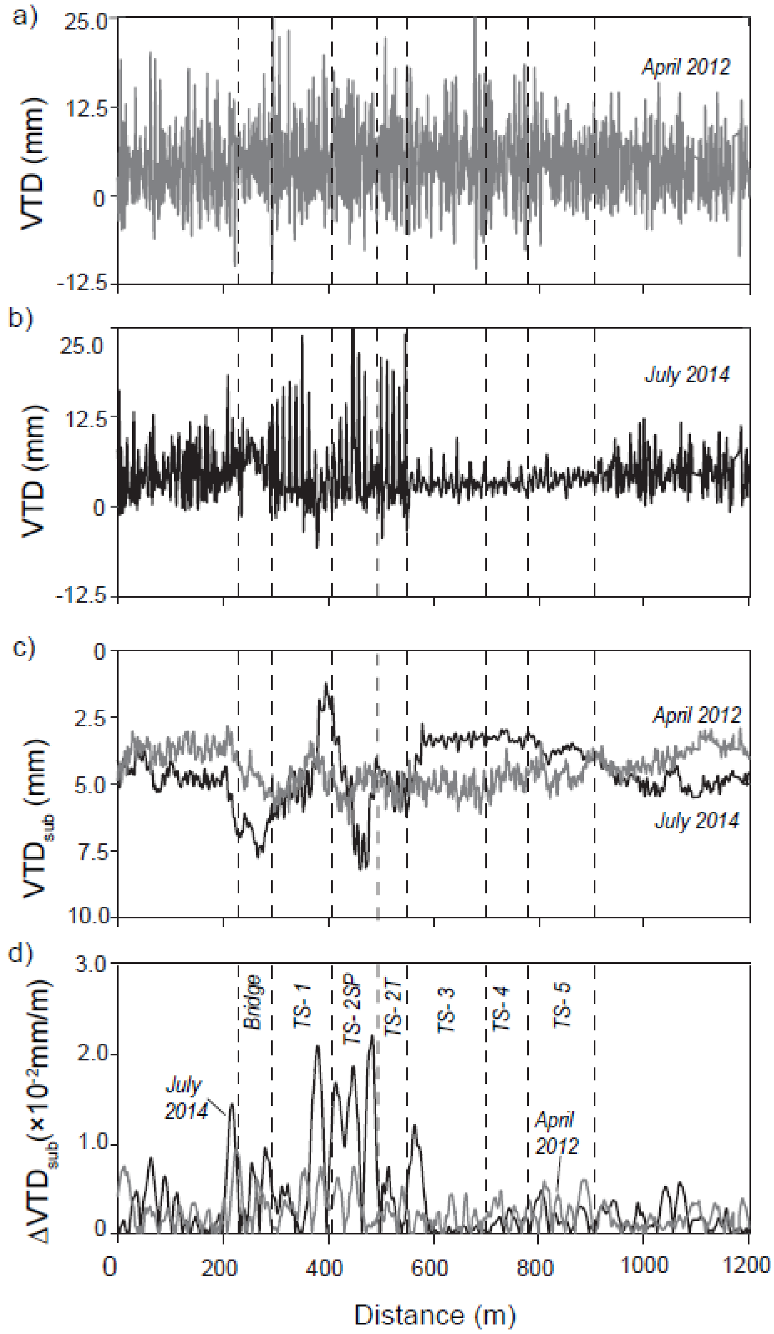

Figure 5-6. Plots of the VTD measurements over the study area measured on (a) April 2012 (before upgrades), (b) July 2014 (after upgrades), (c) the VTD_{sub} calculated with 20 m moving window, and (d) the ΔVTD_{sub} from April 2012 and July 2014 tests.

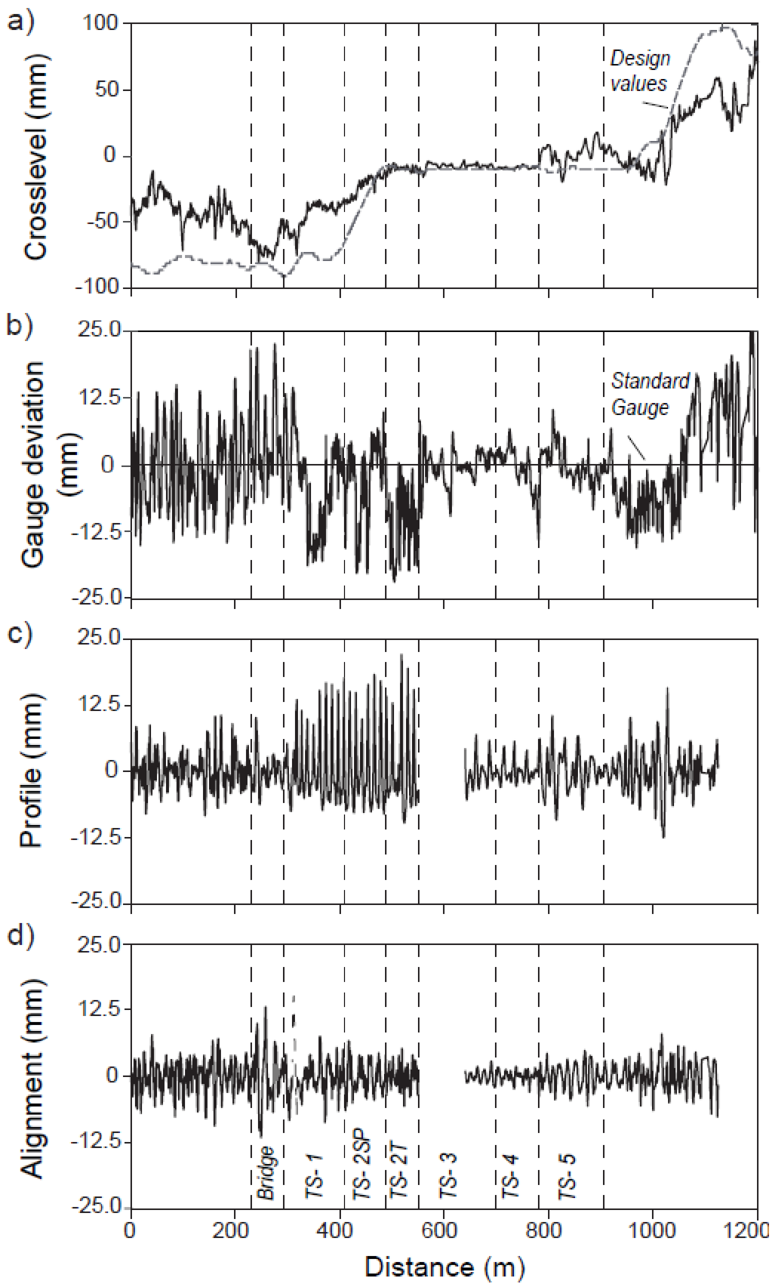

Figure 5-7. Plot shows the variation of (a) the measured and designed crosslevel values, (b) the deviation from standard gauge, (c) the average profile measurement, and (d) the average alignment measurement during the track geometry measurements in 2013.

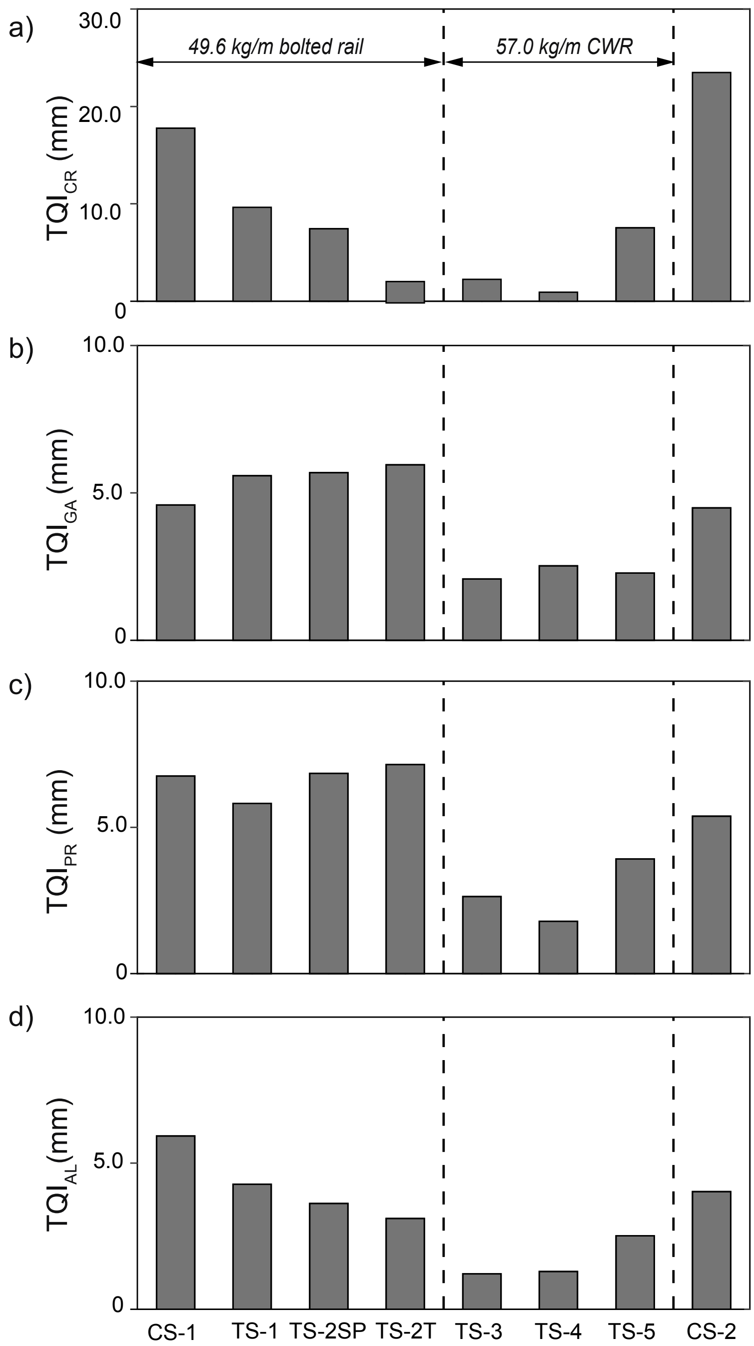

Figure 5-8. Plot shows (a) TQI_{CR}, (b) TQI_{GA}, (c) TQI_{PR}, and (d) TQI_{AL} for each test section during the track geometry measurements in 2013. The TQIs at each test section is equal to the standard deviation of the geometry measurements.

5.8. Comparing the VTD_{sub} values from 2012 to 2014

The impact of differing axle loads of the adjacent car during the VTD measurements were evaluated using both the Winkler model as per Fallah *et al*. (2016), and empirically. The Winkler modelling produced a relationship (Equation 5-3) between VTD_{sub} values measured in 2012 ($VTD_{sub,2012}$) to those measured in 2014 ($VTD_{sub,2014}$). This relationship is plotted in **Figure 5-9a** and it is nearly linear which is a result of the linear elastic foundation used in the model. The model shows a zero intercept, which is necessary as an infinitely stiff subgrade would result in zero VTD regardless of loading configuration.

$$\mathrm{VTD}_{\mathrm{sub,2014}} = 1.05 \times \mathrm{VTD}_{\mathrm{sub,2012}}{}^{1.07}, \qquad \text{Equation 5-3}$$

The empirical relationship for the VTD_{sub} measurements were developed from a comparison of VTD_{sub} values $VTD_{sub,2012}$ and $VTD_{sub,2014}$ between mileposts 500 and 610 that run within valley of the Bad River and has similar geological and subgrade conditions as found at the study site. This comparison of data sets was based on the coordinates from the Global Positioning System (GPS) used for the measurements. The GPS systems used for the measurements have a specified R95 (the radius of a circle centered at the true position, containing the position estimate with probability of 95%) of 3.7 m. Thus, to account for the offset between the location, the entire track between mileposts 500 and 610 was divided into sections of 80 m, over which the average VTD_{sub} were calculated. The 80 m length was selected to be within the same range as the length of test sections and thus comparable to the values calculated for the test sections. The VTD_{sub} measurements from the different years are plotted versus one another in **Figure 5-9b** it is evident that the VTD_{sub} measured during the 2014 run are higher than during the 2012 run due to the higher axle loads on the adjacent car, with the notable exceptions of the test sections. A power regression was fitted to the data plotted in **Figure 5-9b** to obtain an equation (Equation 5-4) for the conversion of the data. The power regression was selected to be consistent with the Winkler model, to allow for the data to fit within the range of VTD_{sub} measured, and to fit to a zero intercept. The coefficient of correlation (R^2) for **Eq. 4** and the data set was 0.8. Other linear and non-linear regressions were tried but did not improve R^2; the linear regressions forced through the origin resulted in a very poor fit. 95% confidence bounds for this fit were also calculated. Equation 5-4 and the confidence bounds are presented with the data in **Figure 5-9b**, and with the Winkler model results in **Figure 5-9a**. The equation for the fit and the 95% confidence bound were found to be insensitive to the length of sections greater than 20 m used to calculate the average VTD_{sub}, shorter lengths resulted in more data points and lower coefficients of correlation.

$$\mathrm{VTD_{sub,2014}} = 2.21 \times \mathrm{VTD_{sub,2012}}^{0.63}, \qquad \text{Equation 5-4}$$

From **Figure 5-9a**, the Winkler model does not provide a relationship that is consistent with the measured data, and this difference is attributed to the assumption of linear elasticity in the Winkler model which has previously been found to not be consistent with field conditions (Zarembaski & Choros, 1981). Thus, the empirical correlation (Equation 5-4**)** was used to convert the 2012 measurements to a value consistent with 2014 loading. The results of this conversion allow for the comparison of the two VTD_{sub} measurements (**Figure 5-10a)**; and, the ΔVTD_{sub} calculated from the converted 2012 data to that calculated from the 2014 data (**Figure 5-10b)**. **Figure 5-10a** shows that the VTD_{sub} values outside of the test sections are very similar, and there are notable changes in VTD_{sub} where the track has been reconstructed. To further simplify the comparison, the average of VTD_{sub} and ΔVTD_{sub} at each test section (presented in **Figure 5-10a** and **Figure 5-10b**) are calculated as the representative of the overall track condition within the section and the results are presented in **Figure 5-11a** and **Figure 5-11b**. The 2014 VTD_{sub} for the control sections (CS-1 and CS-2) are very close the average corrected 2012 VTD_{sub} values and confirms the applicability of Equation 5-4 in the conversion of this data. The 2014 ΔVTD_{sub} and the converted 2012 ΔVTD_{sub} are within the 95% confidence bounds for converted 2012 but only by a small margin, and small variations of magnitudes similar to the confidence interval should be interpreted with caution.

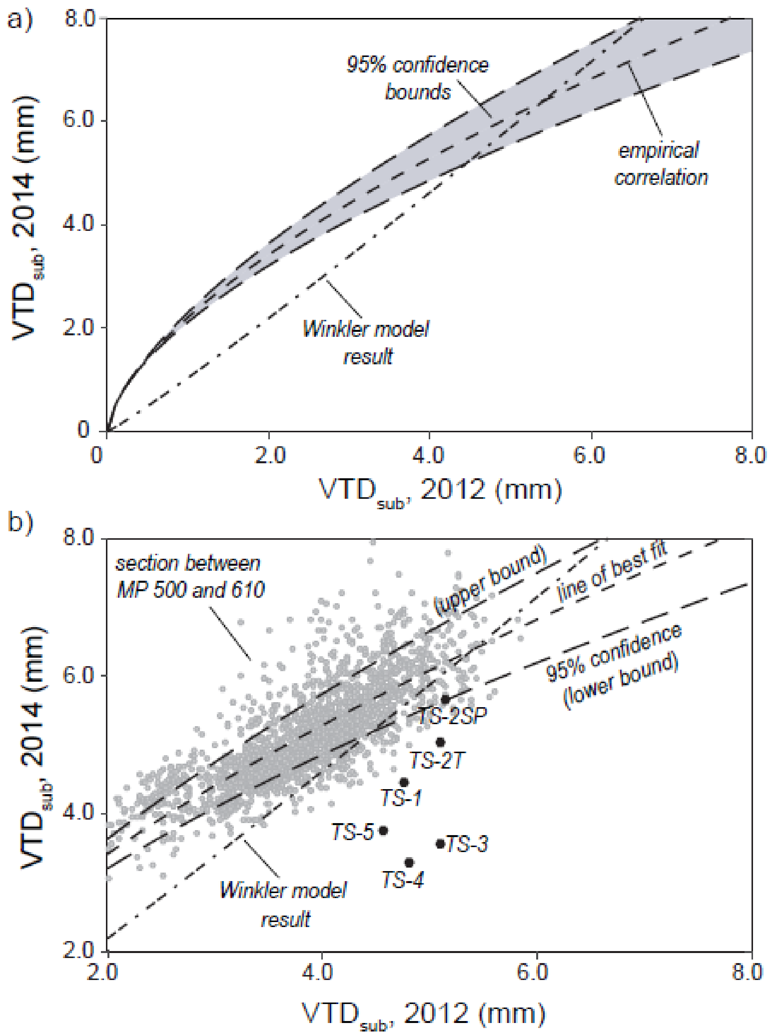

Figure 5-9. Plot showing a) the line of best fit and 95% confidence bounds for the field data, and the conversion generated from the use of a Winkler model and b) VTD_{sub} measured over the PRCS between mileposts 500 and 610 in 2012 ($VTD_{sub,2012}$) versus that measured in 2014 ($VTD_{sub,2014}$).

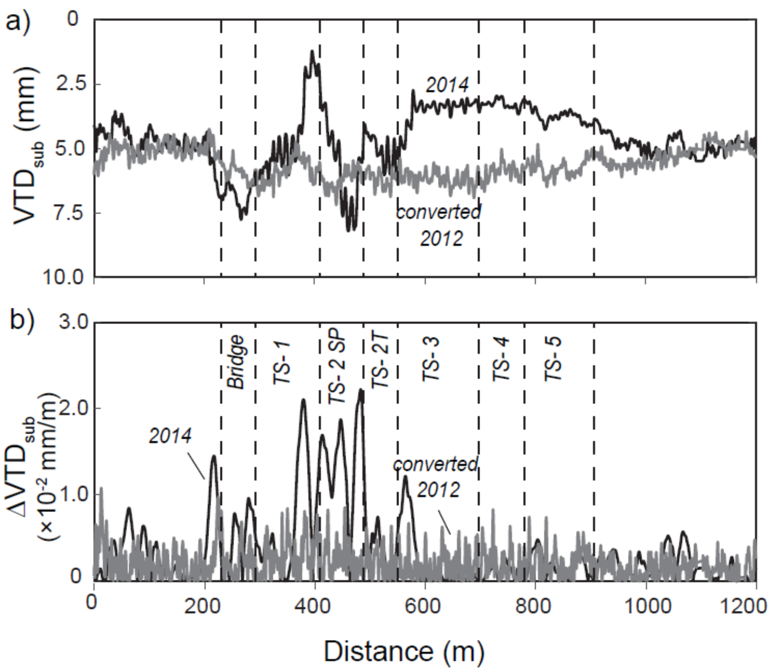

Figure 5-10. The plot showing (a) the VTD$_{sub}$ for 2014 test and the converted 2012, and (b) the ΔVTD$_{sub}$ from 2014 test and the calculated ΔVTD$_{sub}$ based on the converted 2012 VTD$_{sub}$.

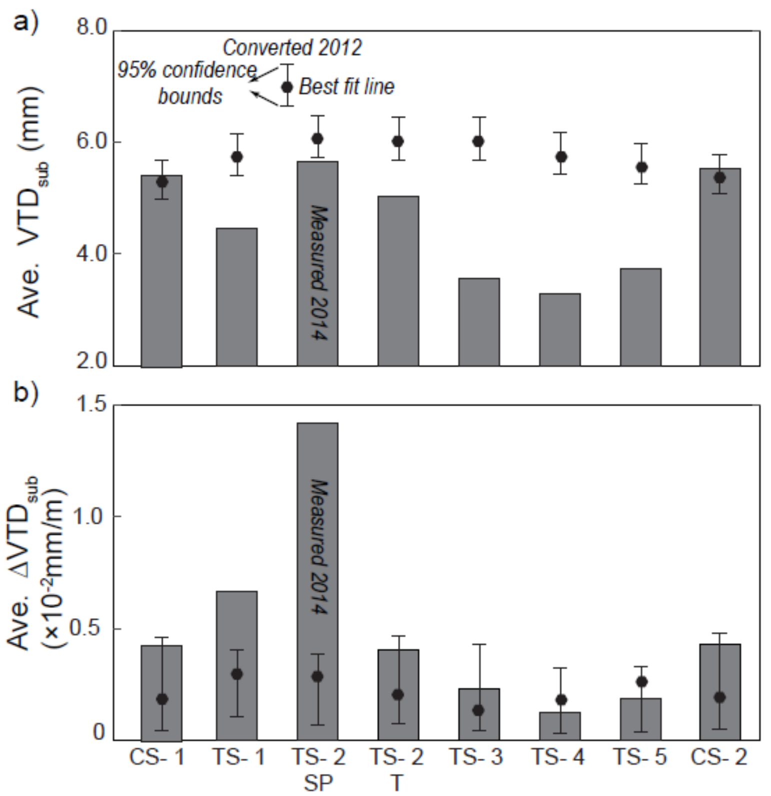

Figure 5-11. The 95% confidence bounds for normalized VTD$_{sub}$ from 2012 run and the measured VTD$_{sub}$ from 2014. CS-1 and CS-2 are control sections that are located 8.0 km before and after the test sections.

5.9. Observed track performance over remediated track

The following section presents the observed performance that resulted from the implementation of the differing methods of upgrading the track. These performances are presented in terms of the VTD parameters and TQI values calculated from the measurements.

5.9.1. Effect of the excavation of soft subgrade and compaction of fill material

The effect of the undercompacted fill material in section TS-2SP is not readily evident in the TQI results (**Figure 5-8**); however, it is clearly visible as a large increase in VTD_{sub}, and a corresponding increase in ΔVTD_{sub} (**Figure 5-10a** and**Figure 5-10b**). Within **Figure 5-11**, this section shows a slight improvement in average VTD_{sub} over the results from the preconstruction measurements, but less than those for adjacent section (TS-2T) with the same embankment construction, similar excavation of poor quality subgrade material and evidence of better compaction of fill material. From **Figure 5-10a**, the VTD_{sub} post reconstruction beneath sections TS-1 and TS-2 are highly variable **(Figure 5-10b** and **Figure 5-11b)** and the average values presented in **Figure 5-10a** are strongly influenced by a very stiff location at the southern end of TS-1 and a very soft location in the middle of TS-2SP.

There is a strong relationship between ΔVTD_{sub} and the development of high track geometry values, and a poor correlation between ΔVTD_{sub} and TQI (Roghani and Hendry 2016b). There is no apparent correlation between the peak values in the track geometry shown in **Figure 5-7**, or the TQI shown in **Figure 5-8** with the location of high ΔVTD_{sub} shown in **Figure 5-10**. This suggests that either not enough time or train traffic had passed to degrade the track geometry to a level that would start indicating track degradation and does not allow for the differentiation of performance between the test sections. Based on the correlations presented in Roghani and Hendry (2016b), high ΔVTD_{sub} suggests that there may be long-term maintenance issues on the southern portion of TS-1 and in TS-2SP that are a result of the implementation and not the design.

The measurements from TS-2SP are excluded from the comparisons between the different test sections as the results do not appear to be representative of the performance that can be attained from the designed structure.

5.9.2. Trend in TQI over length of study site

From **Figure 5-8a**, and **Figure 5-8d** there is a trend of decreasing of TQI_{CR} and TQI_{AL} towards the center of the remediated section for both the sections with jointed rail and CWR. This does not show the effects of the under compacted fill under TS-2SP. The cause of this trend could not be determined from the available data, but may be the results of the track curvature or dynamic responses of the rail cars which are initiated on the sections of track on either end of the study site.

5.9.3. Replacement of jointed rail with heavier CWR

From **Figure 5-8**, it is clear that the sections that have CWR have notably lower TQI_{GR}, TQI_{PR}, and TQI_{AL} values, and thus performance, than the sections of jointed rail. This was expected, and has been previously noted and attributed to the removal of the joints and the deviations of the different geometry measurements at these joints (AREMA, 2012).

The more notable impact of the installation of the CWR is apparent in the section where it was the only improvement made to the track (TS-5), where it halved the average VTD_{sub} over the section (range of 41 to 68% between the 95% confidence bounds) (**Figure 5-11a**). This magnitude of improvement in VTD_{sub}, representing a significant increase in track stiffness, was also evident in the other sections where CWR was installed with other substructure changes (TS-3 and TS-4); the other substructure changes appear to result in only relatively small over what was obtained from the installation of CWR alone.

5.9.4. Excavation of poor subgrade and reconstruction of embankments

The reconstruction of the embankment to AREMA guidelines shows a reduction of VTD_{sub} between 21 and 44% of track deflection (TS-1 in **Figure 5-11a**). This is less improvement in stiffness than that obtained from the installation of the CWR alone. When combined, the effect of both the reconstruction of the substructure and the installation of CWR reduces the track deflection between 58 and 89%.

5.9.5. Replacement of portions of subballast with geogrid

Figure 5-11a shows a relatively small increase in the VTD_{sub} observed over the sections where geogrid was used with 150 mm of subballast versus the sections that used the full 300 mm of subballast. Referring back to **Figure 5-10,** the VTD_{sub} beneath the jointed rail (TS-2T versus TS-1) is highly variable and the results of this comparison are not conclusive; whereas the comparison of the VTD_{sub} beneath the CWR sections show a very consistent deflection (TS-3 with TS-4) and thus no apparent difference in performance.

5.10. Conclusions

This paper has presented a methodology for quantifying the effectiveness of different methods used to improve the railway track performance on soft subgrades. This methodology is comprised of quantifying the changes in track stiffness from before and after VTD measurements, and the evaluation of the roughness of the track that has developed since the track upgrades.

A project was presented as a case study to explain the steps of this methodology. This project consisted of upgrading the track superstructure from 49.6 kg/m (100 lb/yd) bolted rail to 57 kg/m (115 lb/yd) continuously welded rail; embankment reconstruction, and using a layer of geogrid at the subballast - subgrade interface. The VTD measurements were capable of measuring changes in track deflection, and thus modulus, due to the upgrading of the track structures with a high enough resolution to distinguish between the differing test sections. The track geometry measurements showed that the track was still in very good condition across the study site, and there are no locations at which the geometry exceeds the thresholds defined by the FRA for either this class of track, or even the much more stringent thresholds of a Class 5 track (FRA, 2007). The high variation of ΔVTD_{sub} that appeared over a portion of the study site after the embankment reconstruction and the lack of corresponding impact on the TQI suggests that not enough time or train traffic had passed to degrade the track geometry to a level that would start indicating issues and allow for the differentiation of performance between the test sections.

The evaluation of the effectiveness of the different remediation methods showed that the replacement of jointed rail with heavier CWR significantly increased the track stiffness, reducing the VTD_{sub} by 41 to 68%. The excavation of the subgrade and reconstruction of the embankment had a lesser effect on average, of 21 and 44%, though some location showed that it resulted in a very stiff structure (**Figure 5-10**; TS-1). The combined effect of CWR and the substructure upgrades further improved the track modulus. The results of replacing the suggested 600 mm of subballast with 300 mm of subballast and a geogrid were inconclusive under the jointed rail, and showed no change in the performance of the track under the CWR.

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6. Chapter 6: Conclusions & Recommendations

The focus of this research was to provide a greater understanding of the significance and influence of subgrade on railway operations; and, to provide a framework that can be used by industry to map the location of soft subgrade so that they can take measures to increase the safety and the reliability of the railway network. This research is the continuation of a study at University of Alberta initiated following recommendations by the Transportation Safety Board of Canada (TSB, 2008) that *"[t]he railway inspection technologies and procedures, mainly based on evaluations of track conditions at surface level, were not effective to assess the condition and the behaviour of the subgrade and detect the impending risk of collapse."*

The following sections explain the conclusions corresponding to each objective of this research.

6.1. Mapping subgrade condition along a railway line

Chapter 3 presented the methodology developed for the use of continuous vertical track deflection (VTD) measurements from a moving loaded rail car to map the subgrade condition along a railway line. The methodology developed following the collection of 12,000 km of measurements. The examination of VTD data showed that there is a clear difference between the measurements taken on track with continuous welded rail (CWR) and jointed rail, with the large peaks and negative values occuring at a relatively constant interval , about 12 m, in a jointed rail. This distance corresponds to the space between joints in the track. These large peaks and negative values are not uncommon in CWR tracks and are resulted from the inclination of the instrumented truck at joints or any other localized soft location along the track. As a result, the track surface condition impacts the VTD measurements and does not allow for the interpretation of the subgrade condition. The Fast Fourier Transform (FFT) analysis performed to assess the extent of the effect of track surface condition on the VTD suggested that the major contributing amplitudes occur at wavelength between 5.5 and 6.5 m for bolted joints. This is the average space between two staggered joints on alternate rail. The spectrum of higher magnitude and shorter wavelengths extends up to 20 m and is followed by a gap that extends up to wavelength of 100 m. The wavelengths greater than 100 m are of a scale more consistent with the length of features resulting from variations of earth structures, geology, and terrain. To minimize the influence of surface imperfections, moving average, a simple yet effective low pass filter, with 20 m window was employed to remove lower wavelength data resulting from surface condition. The result is a filtered version of the VTD data (VTD_{sub}) that is a measure of vertical track deflection that is predominantly a result of subgrade. The comparison of the VTD_{sub} with substructure and subgrade conditions at different scale, over which the subgrade is changing, confirmed the effectiveness of the filtering procedure and its suitability to identify the soft subgrade locations. Two limitations were also identified with the use of VTD measurements. First, the filtering smoothed the features that occur at wavelengths shorter than the length of filtering. Thus, the interpretation of sharp change requires judgment and knowledge of the site. Second, the load of the rail car adjacent to the measurement system greatly effects the resulting measurements, and thus should be kept constant to ensure measurement repeatability and facilitate the comparison of subsequent measurements. Experience from this study indicates this is very difficult to achieve when the car equipped with the MRail system is shipped in revenue service.

6.2. Quantifying the impact of subgrade stiffness in track performance

Chapter 4 presented the quantification of the impact of subgrade stiffness on the prevalence of track geometry defects and degradation of track quality indices (TQI). The data included comes from two high traffic subdivisions (>50 GMT/year) in Canada with total length of 800 km and consists of vertical track deflection (VTD) measurements and three years track geometry measurements. The VTD measurements were used to derive two indices to quantify the subgrade condition: First, the VTD_{sub} that is representative of the substructure and subgrade condition, and thus track modulus, which was put into context with AREMA suggested track modulus values.

Second, the slope of the VTD_{sub} versus distance, ΔVTD_{sub}, that is representative of track modulus variability. The VTD parameters, VTD_{sub} and ΔVTD_{sub}, were compared with the defect database to quantify its impact on the development of different types of defects. The database included class 3 and class 4 priority and urgent defects and consisted of gauge defects and surface defects including profile, crosslevel, and warp. It was found that there is a strong relationship between gauge defects (both class 3 and class 4) and ΔVTD_{sub}, but not with VTD_{sub}, with the highest number of defects occurring with *high* ΔVTD_{sub}, and over the stiffest track (*good* VTD_{sub}). This was hypothesized to be the result of higher dynamic loads that occur at transitions between differing track moduli with the stiffest track providing less attenuation for impacts.

The analysis of surface defects (both class 3 and class 4) showed that they are strongly correlated with both VTD_{sub} and ΔVTD_{sub}, with the highest number of defects occurring with *high* ΔVTD_{sub} and the *poor* VTD_{sub}. These surface defects appear to be a result of both high deflections (low stiffness), and dynamic loads resulting from the changes in stiffness.

The further analysis of surface defects showed that the highest frequency of warp, crosslevel, and profile defects occur with poor VTD_{sub} and high ΔVTD_{sub} and thus, both VTD_{sub} and ΔVTD_{sub} contribute to the generation of defects, though warp and profile defects show a greater impact of ΔVTD_{sub}.

The comparison of VTD and TQI data showed that elevated TQI coincided with two cases of VTD: the first are locations with elevated VTD_{sub} and ΔVTD_{sub}; and the second are locations with elevated ΔVTD_{sub} but lower VTD_{sub}. The Plots of TQIs versus VTD_{sub} and ΔVTD_{sub} for each foot of track showed that at any given VTD_{sub} or ΔVTD_{sub} there is a spectrum of TQIs values which results in a poor correlation. The poor correlation was attributed to the impact of maintenance activities that obscures the trends in geometry, with poorly performing structures which have undergone recent maintenance may have close to optimal geometry conditions; whereas, very competent track that has not required recent maintenance may have higher variations in geometry. A simpler comparison, conducted by comparing the distributions of the TQIs for track divided into subsets of *good* and *poor* VTD_{sub}; and subsets of *high* and *low* ΔVTD_{sub}, suggested that the trend of higher VTD metrics resulting higher TQI values does exist within the data.

The contrast between the strength of the relationship between defects and VTD but not between TQI and VTD was suggested to be the result of the use of threshold values for geometry in the evaluation of defects, versus analyses of the full spectrum of possible TQI values. Further analyses were conducted by comparing the locations that exceeded an arbitrary threshold of 3.0 mm and the VTD parameters. It was found that there is a strong correlation between TQI_{GA} and surface related TQI with ΔVTD_{sub} and showed no correlation with VTD_{sub}. A sensitive analysis was also conducted on the threshold value for TQI and it was found that the threshold does change the percentage of track at each category; the trends are independent of the thresholds.

6.3. Quantifying the effectiveness of different upgrade methods

A methodology was developed for quantifying the effectiveness of different methods used to improve the railway track performance on soft subgrades. This methodology is comprised of quantifying the changes in track stiffness from before and after VTD measurements, and the evaluation of the roughness of the track that has developed since the track upgrades.

A project was presented as a case study to explain the steps of this methodology. This project consisted of upgrading the track superstructure from 49.6 kg/m (100 lb/yd) bolted rail to 57 kg/m (115 lb/yd) continuously welded rail; embankment reconstruction, and using a layer of geogrid at the subballast - subgrade interface. The VTD measurements were capable of measuring changes in track deflection, and thus modulus, due to the upgrading of the track structures with a high enough resolution to distinguish between the differing test sections. The track geometry measurements showed that the track was still in very good condition across the study site, and there are no locations at which the geometry exceeds the thresholds defined by the FRA for either this class of track, or even the much more stringent thresholds of a Class 5 track (FRA, 2007). The high variation of ΔVTD_{sub} that appeared over a portion of the study site after the embankment reconstruction and the lack of corresponding impact on the TQI suggests that not enough time or train traffic had passed to degrade the track geometry to a level that would start indicating issues and allow for the differentiation of performance between the test sections.

The evaluation of the effectiveness of the different remediation methods showed that the replacement of jointed rail with heavier CWR significantly increased the track stiffness, reducing the VTD_{sub} by 41 to 68%. The excavation of the subgrade and reconstruction of the embankment had a lesser effect on average, of 21 and 44%, though some location showed that it resulted in a very stiff structure. The combined effect of CWR and the substructure upgrades further improved the track modulus. The results of replacing the suggested 600 mm of subballast with 300 mm of subballast and a geogrid were inconclusive under the jointed rail, and showed no change in the performance of the track under the CWR.

6.4. Implication of study

The major contribution of this research is to develop a methodology to use rolling deflection measurements to map the subgrade condition along a railway line. This is valuable information for assessment of track quality that has not been available to industry to date. The methodology can be used for evaluating the capacity of branch lines with respect to proposed increased axle loads and required capital project. Even though this methodology developed based on the measurements form MRail system; its application is not limited to the MRail system.

The second major contribution of this research was to quantify the impact of subgrade stiffness and its variation on the track geometry. The previous understanding of the influence of subgrade on track performance has been obtained through localized field measurements, observations and extensive experience within the industry. In this research, an extensive data analysis, over 800 km of track with a wide variety of subgrade type and quality, were conducted to quantify the extent of the impact of subgrade on prevalence of geometry defects and degradation of track quality indices. The track geometry defects are the second leading cause of derailments in both the United

States of America and Canada and track quality indices affects the extend of the maintenance required for a section of track. This research presented the underlying causes that result in the degradation of track geometry and allows for the identification of sections where it is most likely to occur. This is valuable information to railway industry than can reduce the risk of train derailments and greatly improve the safety of track.

6.5. Recommendations

The work presented in this thesis provided a greater understanding about the impact of soft subgrades on the performance of the railway tracks and the methods that can be used to enhance the track quality. The following tasks are recommended for future works:

- Improving the configuration of the system: The VTD from the MRail system showed that the measurements are impact by the inclination of the instrumented truck. This inclination leads to large peaks or negative values at the localized soft location such as joints. A detail analysis needs to be conducted on the configuration of the system to develop a means to obtain a non-relative measurement, and thus, to minimize the impact of the inclination of truck. This may include adding more lasers and/or cameras or changing their locations.
- Improving the filtering of VTD measurements: In this research, the moving average with 20 m window length was used as a low pass filter to eliminate the impact of surface conditions from VTD measurements. The 20 m is the minimum length of filtering that is applicable to any section in a jointed track, however, some sections may allow for a smaller window, thus higher resolution can be achieved. A program can be developed to determine the required window length for any specific section of track. This will help to keep as many feature as possible within the filtered data. In addition, a sensitive analysis can be conducted to compare the effectiveness of different low pass filters at different filtering length.

- Identifying different types of defect from VTD measurements: A Fast Fourier Transform analysis can be conducted to understand the extent of the impact of different types of defect on the VTD measurement with respect to magnitude and wavelength and evaluate the potential of VTD measurements for identifying the location of defects on a railway track. This would be valuable for railway industry as a great amount of information can be obtained regarding the track quality and performance with a just single run of a rolling deflection system.
- Using VTD measurements for evaluating the bridge condition: This study was focused on using VTD measurements to evaluate the subgrade condition. The results showed that the measurements are strongly correlated with the condition of track foundation. Therefore, these measurements have the potential to be used for assessing the bridge condition. This requires an extensive analysis of field data to determine the extent of the impact from different bridge components on the VTD measurements. This information can be used to develop a framework for assessing the bridge condition over time.
- Standardizing the testing condition: The result of measurements showed that the load of adjacent car has a great impact on the measurements. Experience from this study also indicated that this is very difficult to control the loading from the adjacent car when it is shipped with revenue service trains. Therefore, in order to get comparable datasets, it is recommended to run the instrumented car along with the track geometry car. This has the advantage of maintaining similar loading condition and comparing the VTD and track geometry from same test runs.
- Quantifying the impact of subgrade on rail break: Rail break is one the reasons leading to derailments in cold regions like Canada. The impact of subgrade on the frequency of rail breaks has not been studied. The framework presented in this thesis regarding quantifying the subgrade condition can be used to study the correlation between the rail break and subgrade condition.

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A. Appendix A: A case study of the assessment of an existing rail line for increased traffic and axle loads

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A.1. Summary

The demand for reliable, efficient and economical rail transportation has increased significantly over the past decade. In order to meet these needs, the rail industry is working to improve its operational capacity through increasing axle loads. These increased loads often exceed what these lines, and specifically branch lines, have been previously been subjected to. A major challenge in the planning for a large increase in the trainloads is the assessment of the current track and subgrade conditions from which the source of problems are identified and the determination of what upgrades are required. Canadian National Railway's (CN) Lac La Biche subdivision is an example of a rail line over which increases of both axle loads and number of trains is planned. An innovative and large-scale investigation was initiated by CN to assess the current condition of the line, and to estimate the performance under the proposed higher axle loads. This investigation included the use of terrain geological mapping, ground penetrating radar, falling weight deflectometer, the measurement of track deflection under moving heavy axle loads, and shallow sampling of the substructure material. Track geometry car and maintenance records over several years were also integrated in the study to highlight potential problem areas and refine the diagnosis. The analysis of all this inter-related data has allowed for a clearer perspective on the

condition of the track, the source of track problems and recommendations for its improvement. This paper presents the extent of the investigation, the preliminary results, and the challenges encountered in the application of this information for assessment of this infrastructure.

A.2. Introduction

The demand for the transportation of goods by rail in Canada has been increasing significantly over the last decade. Much of this demand has been in the transportation of bulk goods, such as crude oil, from remote locations. These remote locations are serviced by branch lines that may have low allowable axle loads, have only been subjected to light traffic, and traverse challenging terrain. This combination has resulted in the need to upgrade several branch lines to handle mainline axle loads 130 tonne (286 kips), and the expected increase volume of traffic.

The challenge for the rehabilitation of a branch lines is that there is limited knowledge about the history of the performance or construction of the track as they have not been as rigorously monitored and maintained as mainline tracks, and they often have had multiple owners. This makes the suitability of some of these lines for increased axle loads difficult to predict

Thus, the planning for the upgrade of a branch line requires extensive investigation. Canadian National Railway's (CN) Lac La Biche subdivision (LLBS) is a prime example of a branch line on which significant increases of traffic are sought and for which an assessment of the amount of resources to ensure the safe operation of heavier and more numerous trains is required. The LLBS connects the rapidly growing oil producing region around Fort McMurray, in north eastern Alberta, to CN's main east-west line. The LLBS is the first of several CN branch lines that are to be upgraded, and there has been some latitude to explore different methods of investigation to ensure that the information being collected is the best available.

This paper presents an innovative and large-scale investigation of the LLBS. This investigation employed several different available methods to assess the current condition and performance of the LLBS; so as to determine where, how and to what extent, the line needs to be upgraded.

A.3. The Lac La Biche Subdivision

This terrain through with the LLBS was constructed has low to moderate relief, with extensive flat-lying (i.e. poorly draining) areas. This terrain and the colder climate result in expansive muskeg (swamp) areas, and thus the track has long sections with very soft organic subgrades.

These poor subgrade conditions have made the LLBS challenging to maintain over time. The line was set to be abandoned in by its previous owners in 2007 due to the requirement for extensive rehabilitation.

Since reacquiring the line in 2007, CN has heavily invested in improvements to rehabilitate the line for the transportation of materials needed to develop the oil production facilities in Fort McMurray. This rehabilitation included increasing the weight of rail, extensive tie replacement, and increasing the depth of ballast (**Figure A-2**).

The LLBS is presently limited to carrying 121.5 tonne (268 kips) carloads, trains lengths of 90 cars; and typically runs one to two trains per day. CN is considering plans to increase the allowable carloads to 130 tonne (286 kips), lengthen trains, and run up to ten trains per day in order to meet the needs of the growing resource industry in this region.

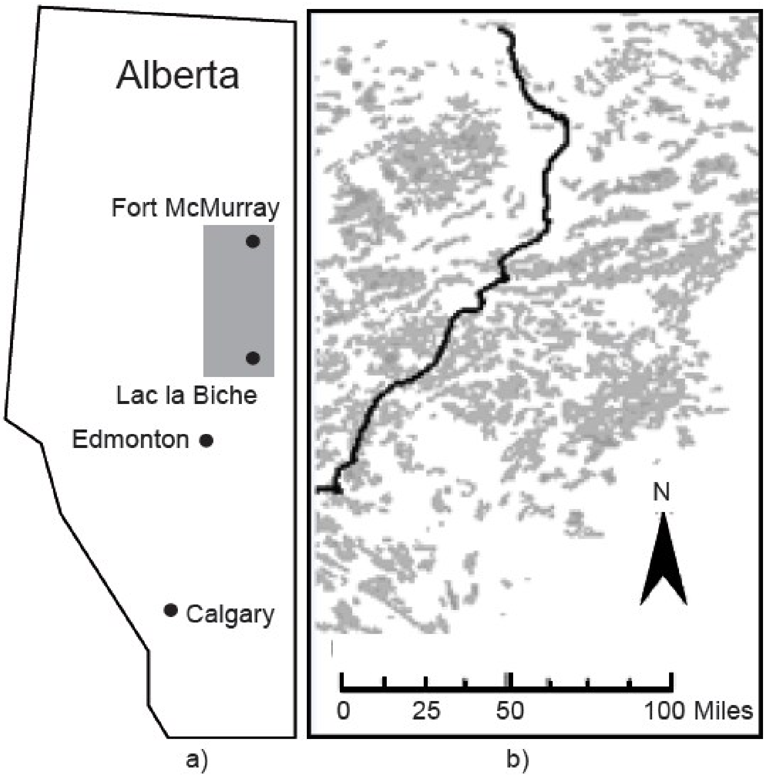


Figure A-1. (a) Location of the study area within Alberta and (b) muskeg terrain along the LLBS

Figure A-2. 3.1 m (10’) ties used to improve track performance at muskeg terrain (Bourgonje and Diercks, 2011)

In mid-2009, it became apparent that adding ballast over some of the poorest areas was only effective in the short term (Bourgonje and Diercks, 2011). CN in collaboration with Loram proceeded to use a lifter machine to provide a 0.15 to 0.2 m (6 to 8) lift of granular fill (Bourgonje and Diercks, 2011) (**Figure A-3**). The purpose of the increased ballast depth was to further spread the load to reduce the stress on the subgrade. Qualitatively, these remediation methods were observed to improve the track performance.

From this experience it was determined that further investigation of the track structures is required before large changes in the train loading were implemented.

Figure A-3. High track embankment due to the cycles of track sinking and lifting

A.4. Investigation of the LLBS

An extensive investigation of the composition and performance of the track structure, over the entire length the LLBS, was conducted in October of 2013. The techniques employed included: terrain analysis, a track deflection measurement system; ground penetrating radar (GPR); continuous lidar and photograph surveys, falling weight deflectometer, and physical substructure sampling.

A.4.1. Track Deflection Measurement (MRail)

The measurement of the deflection of the track structure under moving axle loads was conducted with the 'MRail' system. The MRail system was originally developed at the University of

Nebraska at Lincoln in collaboration with the (*United States*) Federal Railroad Administration (FRA) (Norman et al., 2004, McVey et al., 2005, Arnold et al., 2006, Greisen, 2010). The MRail system uses cameras and line laser mounted to the truck assembly to measure changes in the rail at a distance of 1.22 m (4') from the inboard axle (Y_{rel}). The measurements are taken at 305 mm (1ft) intervals along the track along with GPS coordinates of each measurement. **Figure A-4**a and **Figure A-4**b show a schematic and photograph of the MRail system used for mapping of low stiffness track over the LLBS.

The MRail system has seen a slow uptake by the North American industry as a means to identify track defects that are more readily apparent under loading. Research at the University of Alberta (UofA) has focused on the use of the MRail system to map the locations, extent, and severity of soft track structures that are prone to poor perforance and may result in the over stressing of rail. The use of the MRail system on the LLBS was thought to be useful to map out the poorest sections of track in need of upgrading, particularly that over muskeg. This mapping required a loaded rail car, the purchase of an MRail system, and the development and verification of new data analysis and filtering methods by the UofA.

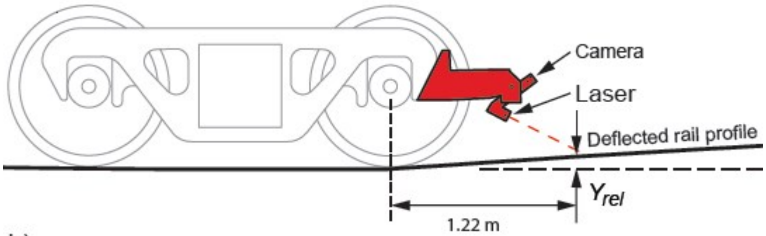
A.4.2. Falling Weight Deflectometer

The Falling Weight Deflectometer (FWD) provides a direct, discrete measurement of trackbed stiffness rather than a general indicator of system stiffness. This technique has been used extensively to evaluate performance of paved highways. The equipment adapted for railbed testing consists of a large weight, approximately 0.5 tonne, which is dropped from a height determined automatically by FWD to give the required load on a beam which transfers the resulting impulse load to a tie. The tie is disconnected from the rail prior to testing so that in effect it acts as a load platen. **Figure A-5**a shows a simplified schematic of FWD measurement and the arrangements of geophones along with the system used for testing in LLBS investigation.

A load cell on the centre of the plate measures the resulting impact and the velocity transducers are used to determine surface velocity at various distances. The velocities are integrated to give vertical displacements. The maximum deflection of the track at 300 and 1000 mm from the drop (d300 and d1000 as shown **Figure A-5**a) gives an indicative value for the deflection of the granular layer (ballast and sub-ballast) and the subgrade, respectively.

The resulting load pulse is similar to that experienced by the tie during the passage of single axle. The drop height can be adjusted to simulate the maximum axle load for the particular railroad being tested. For the LLB testing the target peak impulse load was 14 tonnes which simulates an axle load of 36 tonnes.

a)



b)

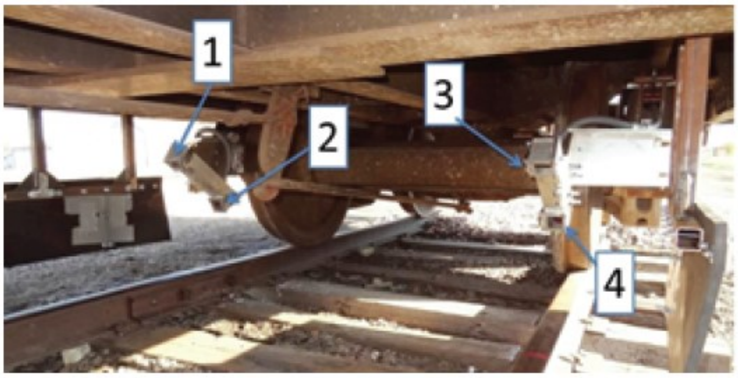


Figure A-4. Plot showing a) schematic of MRail measurement systems and b) MRail system used in LLBS investigation with the location of cameras (1 and 3) and lasers (2 and 4)

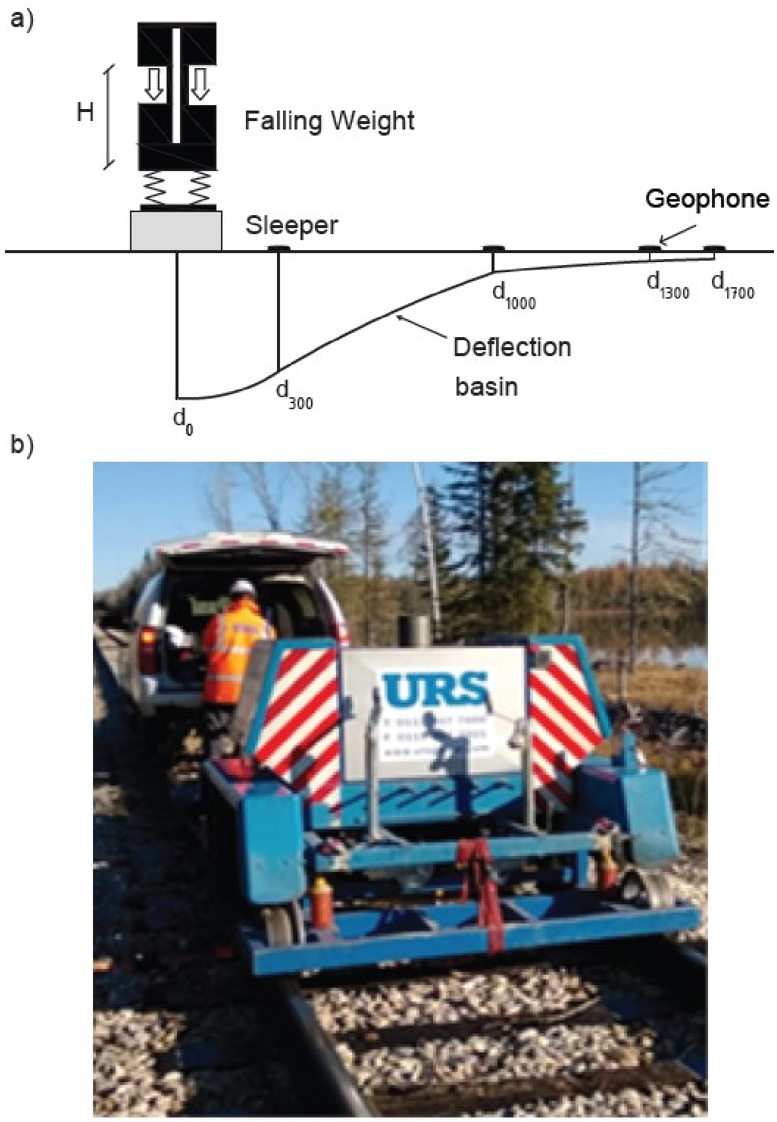


Figure A-5. Plot showing a) Simplified schematic of an FWD measurement system (*after* Burrow et al., 2007) and b) the FWD testing equipment used in LLBS investigation.

Response of track and roadbed under the impulse load are measured using an array of geophones at distances of 0.3 m, 1 m and 2 m respectively, whose output is integrated to give displacement. Both load and displacements are recorded for up to 120ms. **Figure A-6** shows typical time histories of data. Normally only the peak deflections are reported, but data can also be processed to give the velocity of surface wave propagation along the track, the so-called "critical velocity". Fuller descriptions of the techniques are given by Brough et al (2013) and Sharpe et al (2014).

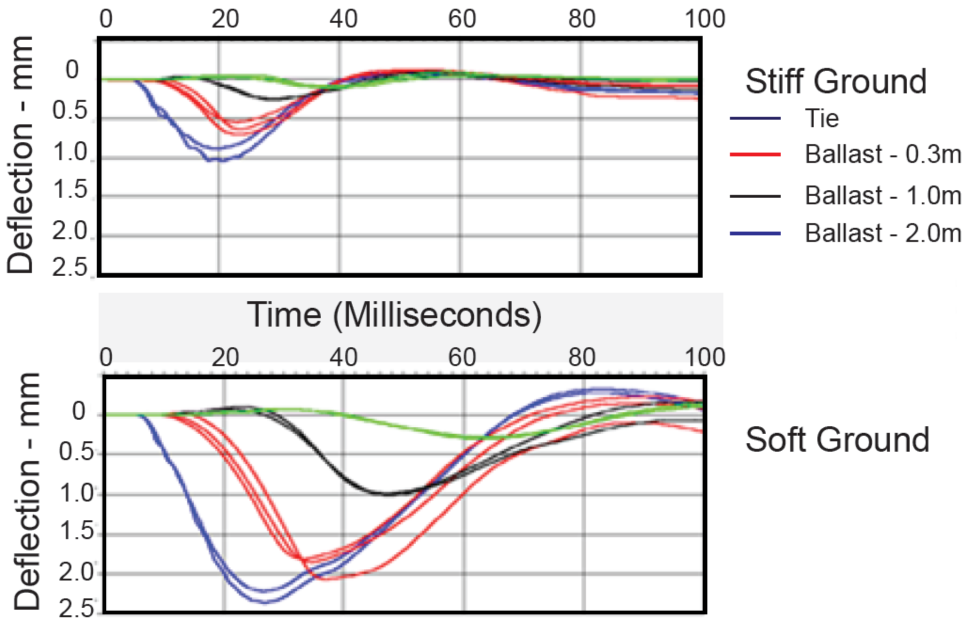


Figure A-6. Typical FWD field data for soft and stiff Trackbeds

A.4.3. Automatic Ballast Sampler

The Automatic Ballast Sampler (ABS) is a system for taking samples of the roadbed and subgrade. The equipment consists of a lined steel sampling tube with cutting shoe.

This is driven typically to a depth of 1.0 m (3ft). If required, ABS can be driven to a total depth of 1.8 m (6ft) by using an extended thin sampling tube. The samples are sealed in the plastic liner, labelled and placed in a storage rack, enabling them to be returned to the laboratory for logging, photography, materials and chemical testing, as necessary. Figure A-7b shows two samples taken from LLBS. Samples were typically taken at a spacing of 160 m (frequency of 10 per mile), as well as at discrete locations as necessary. A total of 287 samples were taken. After sealing the samples were then taken back to the UofA for logging and materials characterization.

a) b)

Figure A-7. Figure shows a) taking sample at LLBS and b) two samples taken from LLBS

A.4.4. Ground Penetrating Radar (GPR)

GPR was used to obtain a continuous measure of the construction of the embankment structures, and the condition of the materials within these structures over the length of the LLBS. In this method, pulses of radio energy are transmitted into the substructure and the returned signals that have reflected off boundaries between substructure layers with different electromagnetic properties are measured (Hyslip et al, 2005). The GPR survey covered the 280 miles of track between Fort McHenry and Edmonton, Alberta and was performed by HyGround Engineering over two days in the fall of 2013. The GPR survey equipment included three pairs of 400 MHz antennas and SIR-20 control units, including collection of laser-based ground topography, digital video and GPS location information.

The GPR data was interpreted to evaluate the thickness and variation of material layers and configuration, a relative moisture content, and a measure of how fouled the ballast is. The variation of ballast and subgrade layers was interpreted to determine if there has been large local subgrade deformation, and if the deformed subgrades have created depressions that retain water.

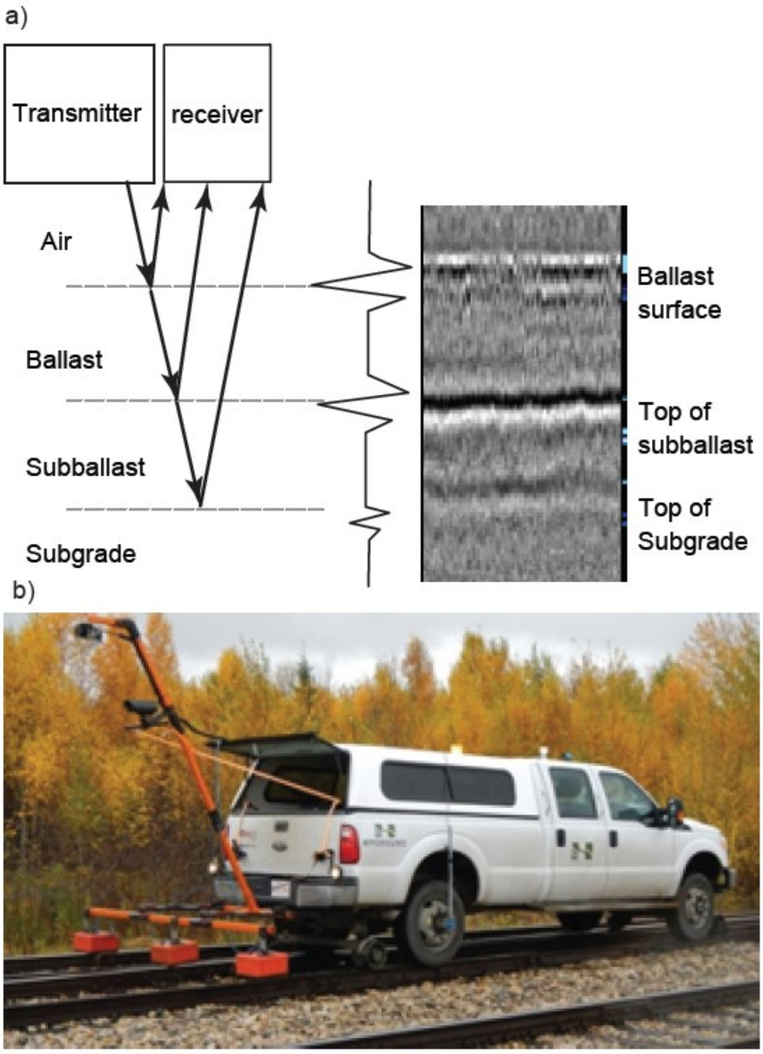


Figure A-8. a) Schematic of GPR for track substructure testing (after Li et al., 2010) b) HyGround's Ground Penetrating radar (GPR) surveying system on CN's LLB Subdivision

A.5. RESULTS

Information provided in this section is a summary of preliminary result of desk study conducted by each part of the collaborative project. Once all datasets are processed, the results will be combined together to conclude the condition of each sections of track along with the recommendations for appropriate remediation.

A.5.1. MRail

Figure A-9 shows unprocessed MRail measurements over 160 m (0.1 mile) of the LLBS. In this section, joints are used to connect the rails. This plot clearly shows the high local Y_{rel} values at joints locations.

The preliminary analysis of data showed that MRail measurements at each point are impacted by the presence of local defects. This is an asset for to find track defects and localized soft spots such as defected/unsupported joints, hanging ties, etc. However, these impacts obscure the influence of subgrade on local measurements. To allow for assessing subgrade condition, a filtering procedure was employed, and validated with an analysis of the terrain and multiple years of track geometry data.

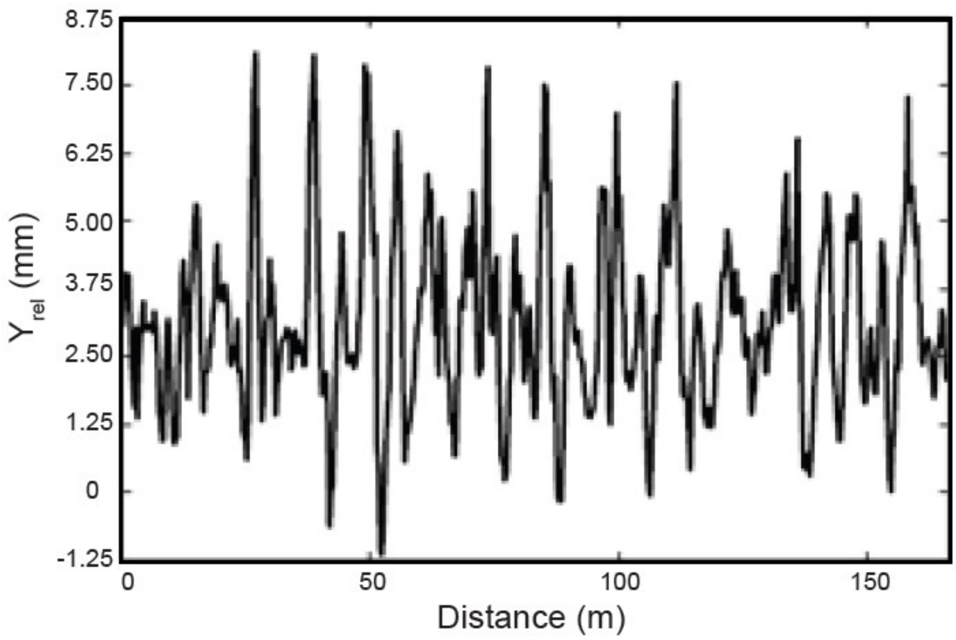


Figure A-9. Plots of the raw MRail data obtained from the LLB subdivision

The results of the use of the MRail system on the LLBS show that it is very useful to map the location of soft subgrade. An example of the results are presented in **Figure A-10** with the colours are used to differentiate the track, and subgrade stiffness. The threshold values for each of the colours are arbitrary. Red represents the softest 5% of the track, orange represents the range from

5% to 15%, yellow represents the range from 15% to 40%, and green represents the stiffest 60% of the track. For More information about the filtering, refer to Roghani and Hendry (2015).

A.5.2. FWD and ABS

Given the large distances of the LLBS that was assessed with the discrete FWD and ABS systems (about 25 miles), the spacing of the test locations was 30 tests per mile. This allowed up to 3 miles of track to be tested per day, and gave a fair characterization of the trackbed. Additional testing was targeted with track supervisor's advice as well as at transitions sections such as bridge approaches and adjacent to the ends of sections with longer ties. It should be noted that sampling was scheduled to provide enough information about the trackbed stratigraphy at FWD testing location. An example of the output of FWD along with taken samples at LLBS is shown **Figure A-11**.

The testing results were aggregated to produce a series of design charts to evaluate the track conditions with respect to the proposed axle loads and traffic (**Figure A-12**). If the track structure and substructure seem inadequate the chart could be used to calculate the required rail section to ensure that ballast strain levels are below the relevant performance threshold. Two performance thresholds are used in developing the proposed design charts: 1) the upper threshold, which represents the "just maintainable" track; and 2) the lower threshold, which represents the aspirational "good track quality", where track geometry should be sustained without the need for more frequent and costly maintenance intervention.

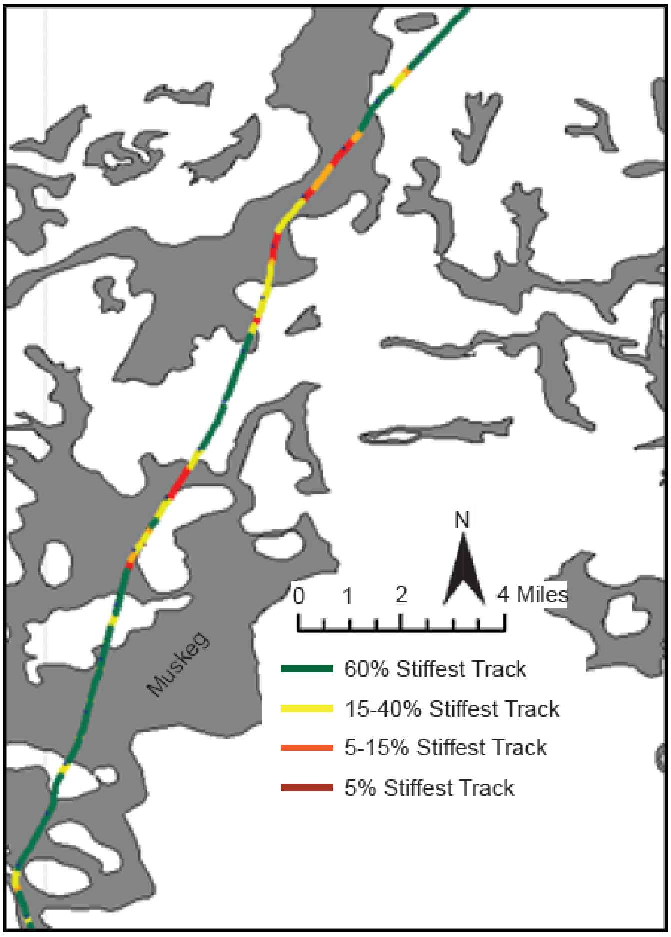


Figure A-10. The map of subgrade quality along LLBS along with the muskeg terrain

A.5.3. GPR

The GPR data was managed and analysed using Roadscanners' Railway Doctor software. The quantitative analysis results from the GPR data consisted of substructure layer thickness, moisture anomalies, and degree of ballast fouling. The GPR data was integrated together in Roadscanners' Railway Doctor software with other data from the LLB subdivision, including: video, mapping, topography from lidar data, track geometry data, MRail stiffness data, and Falling Weight Deflectometer (FWD) data from URS. A plot showing an example of the results is presented in **Figure A-13**.

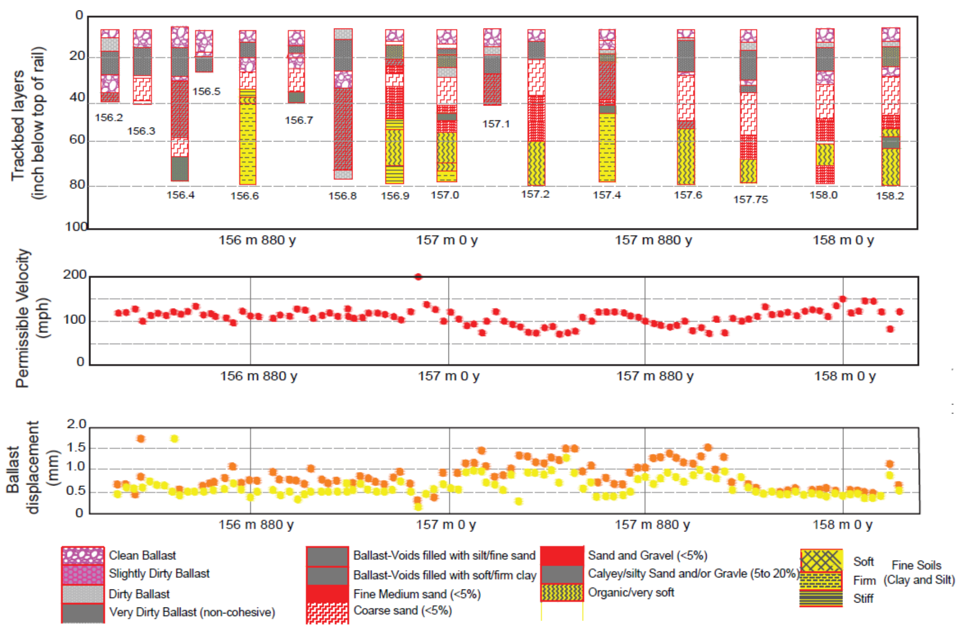


Figure A-11. Example of output of FWD and ABS testing

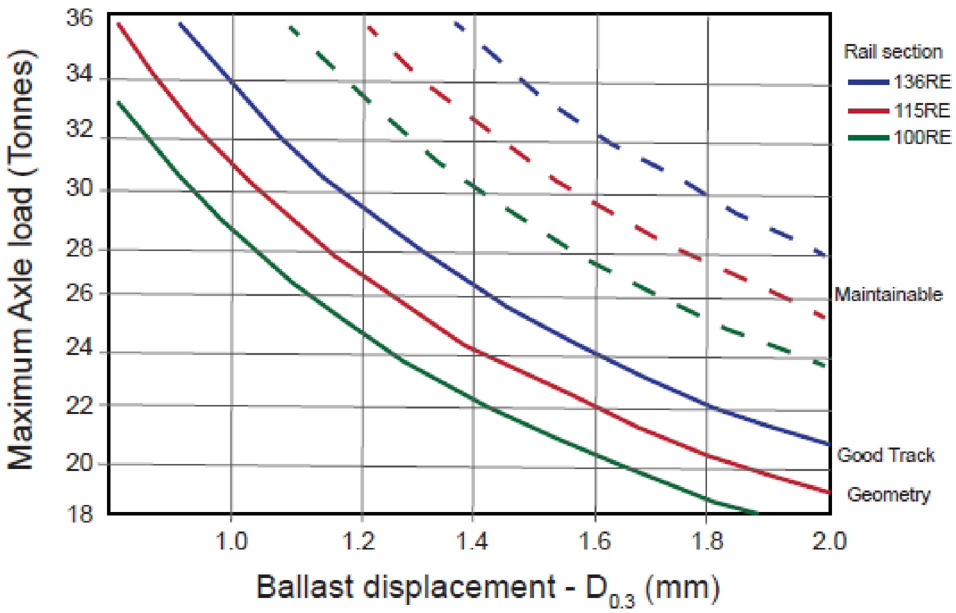


Figure A-12. Proposed design chart for assessing required rail section based FWD measurements

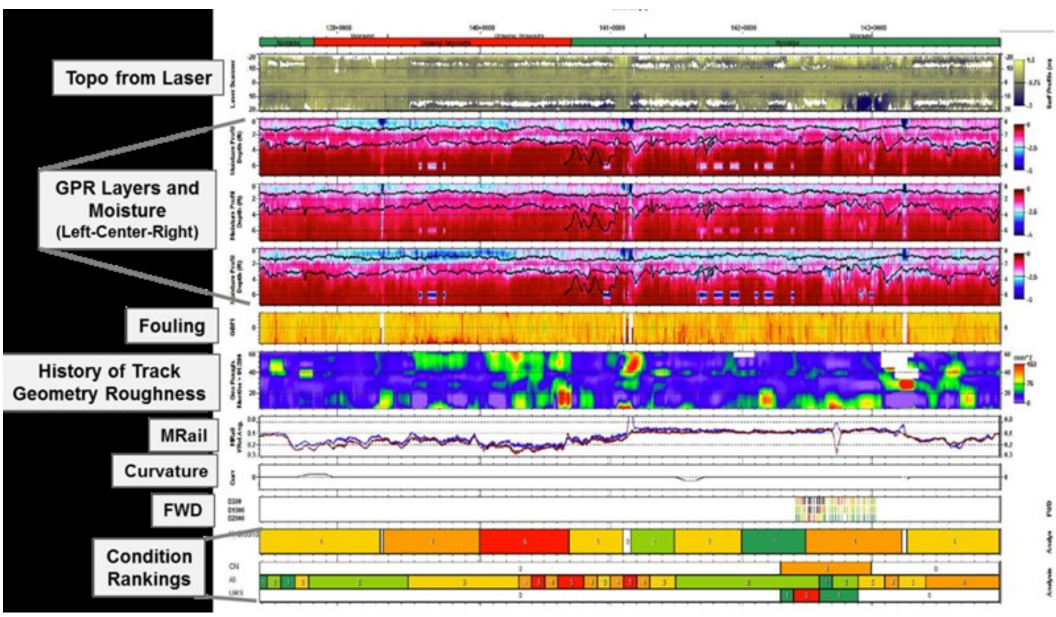


Figure A-13. An example view from Railway Doctor software showing approximately 5-miles of data from the LLB subdivision

A.6. Conclusions and discussion

The investigation of the LLBS was unique in terms of the number of methods employed and the length of the line investigated. This assessment of the LLBS CN has proceeded with the following steps:

I. A thorough field substructure investigation consisted of Terrain mapping, GPR, MRail, FWD, and Multi-runs of track geometry car

II. An analysis of spatial correlation of the results to develop an understanding of track condition at all section of the track. Including of potential weakness such as fouled ballast, weak subgrade, ballast/water pockets or deep soft ground (peat)

III. And finally, the development recommendations for the rehabilitation and upgrading of the track structure specific to the identified weaknesses along the track.

In practice the first step was the simplest, with all of the groups involved in the investigation conducting the work that they do best. Steps 2 and 3 have proven to be the most challenging. The unprecedented amount of data collected has required the development of an equally unprecedented methodology for the evaluation of track condition. Each technology has provided a rating of track quality, and often the ratings differ based on what behaviours or conditions that the testing method identifies.

In spite of the data processing challenges, an assessment of track quality and recommendations of track improvements have been, or are being developed for every 61 m (200 ft) section of the LLBS. Subsequent MRail runs are planned after the start of the implementation of these track improvements, and the track geometry measurements will be monitored over the course of the improvements. This monitoring will allow for the assessment of the effectiveness of the improvements made to the LLBS, and for a review of the weighting applied to the ratings developed from each of the investigation techniques to ensure that it is optimized for future use.

A.7. Acknowledgement

The authors would like to acknowledge Canadian National Railway for providing the project and information for this paper and Canadian Rail Research Laboratory (CaRRL) (www.carrl.ca) for providing lab facilities and students to assist in the on site testing and processing of samples. A special thanks Melissa Ruel from CN.

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B. Appendix B: Combining track quality and performance measures to assess track maintenance requirements

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B.1. Abstract

The serviceability of a section of railway highly depends on track stiffness and roughness. Railway operators regularly measure parameters associated with track stiffness and roughness to evaluate the track conditions. These measures are used in combination with performance observations to assess maintenance requirements. Although these assessments are mostly qualitative, railway operations have benefited from them. Railway operators keep comprehensive records of different types of track defects along their lines. These records are a measure of track performance. They bring an opportunity for quantifying the relationship between track quality and performance. This brings the possibility to develop a performance-based approach for assessing the maintenance requirement along a railway track.

In this paper, a database of track geometry defects along Canadian National's Lac La Biche subdivision (Alberta) has been compared against measured parameters associated with track roughness and stiffness. The analyses confirm the relationship between track stiffness and roughness and the occurrence of track defects. This relationship is further used to define threshold values of track roughness and stiffness, and propose a hazard chart for maintenance requirements along the Lac La Biche subdivision.

B.2. Introduction

Stiffness of track foundations and smoothness of track profiles are two important factors for the adequate performance of a railroad. Soft foundations are associated with increased rail wear and degradation of track components (1, 2, 3). Rough tracks are associated with large dynamic wheel-rail contact forces that can increase the rate of track deterioration (4). Maintaining adequate track stiffness and smoothness is critical for the safety of railroad operations. This is achieved through ongoing track monitoring and maintenance, which comes at great expense. Class 1 railroads in North America (line haul freight railroads with operating revenues of $433.2 million or more (5)), spent a combined total of $7.52 billion on track maintenance in 2011 (6). Railway operators are aware of these maintenance costs; however there is no industry standard for allocating maintenance resources along a railroad. Railway operators regularly measure parameters associated with track stiffness and roughness to evaluate the track conditions. These measures are used in combination with performance observations to assess maintenance requirements in a qualitative or semi-quantitative manner.

This paper proposes a performance-based approach for assessing the maintenance requirement along a section of railway in Alberta, Canada. Records of track geometry defects (track defects) are compared to measurements associated with track roughness and stiffness to establish the relationship between track quality and performance. This relationship is further used to define threshold values of track roughness and stiffness, and propose a hazard chart for maintenance requirements along the section analyzed.

B.3. Study area

The Canadian National Railway's (CN) Lac La Biche subdivision (LLBS) is a branch line connecting the cities of Edmonton and Fort McMurray in the Province of Alberta, Canada (**Figure B-1**). The 274-mile subdivision was constructed over a wide range of subgrade materials, from peat to glacial clay, with a large section lying on soft subgrades. This line is used to haul different types of commodity including crude oil, grains, coal, and etc. The volume of traffic along this line is significantly increasing and it adds to the necessity for a proactive maintenance planning and track upgrades.

B.4. Track quality Measurements

The quality of a railroad track can be measured by different parameters. Some of these are related to surface track roughness, horizontal roughness, track stiffness and gauge. Surface track roughness measurements are common for the assessment of track quality and maintenance planning. Track-geometry cars (such as the one shown in **Figure B-2**) allow continuous measurement of geometric parameters used to calculate track surface roughness. CN runs its track geometry car, shown in **Figure B-2**, over the LLBS 2 to 3 times each year.

Researchers and railway operators have developed approaches to aggregate and evaluate track geometry measures over varying lengths of track (7, 8, 9, 10). Aggregation is done by statistical point-measures (Mean values and Standard Deviation), fractal analyses, and metrics of relative roughness (11, 12). In this study, the Standard Deviation of track geometry measures is used as an index to evaluate track surface roughness. **Figure B-3** shows the variation of this surface roughness index along LLBS between 2010 and 2013.

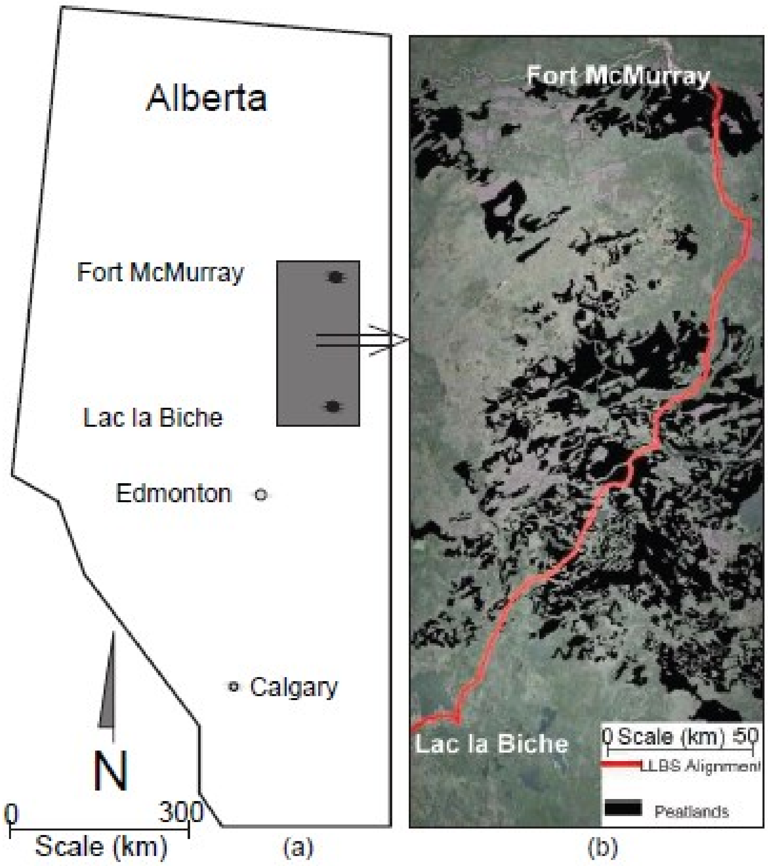

Figure B-1. Location of the study area within Alberta (a) and surface deposits along the LLBS (b) that consist mostly of muskeg (peat)

Figure B-2. CN's track geometry car

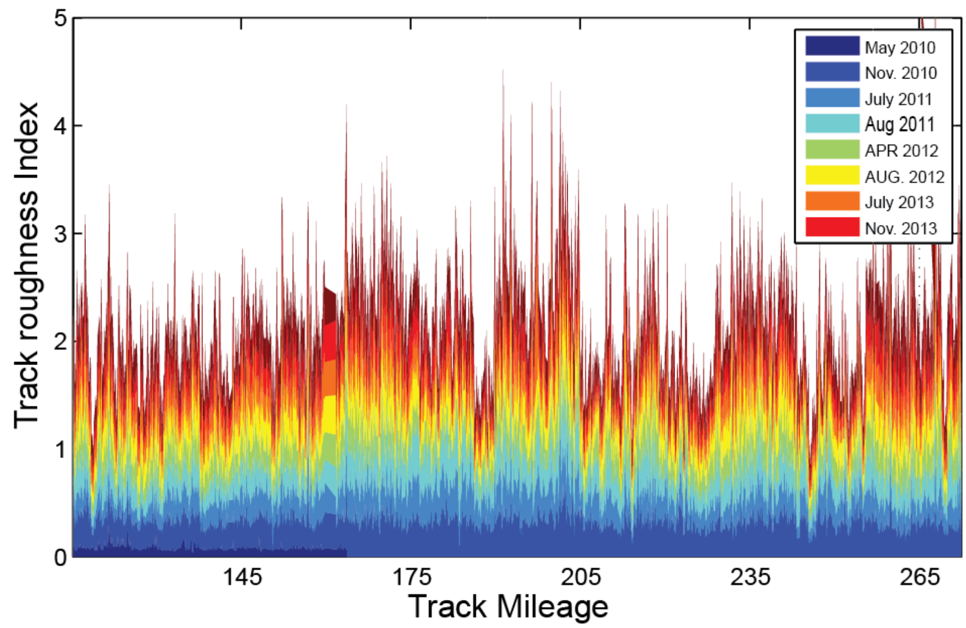

Figure B-3. Variation of surface roughness index along LLBS between 2010 and 2013

Track stiffness is the ratio of track vertical deflection to the applied axle load. Low track stiffness has been related to substructure problems (13). Continuous measurement of track stiffness over long distances has the potential to be a significant addition to the monitoring tools available to track supervisors (14, 15, 16). Some methods to measure track stiffness have been developed based on track deflection measurements while a loaded car is moving on the tracks (17, 18, 19, 20, 21). The University of Nebraska at Lincoln has developed such a system to continuously measure track stiffness (21, 22, 23, 24). Its principles of track deflection measurement are shown in **Figure B-4**.

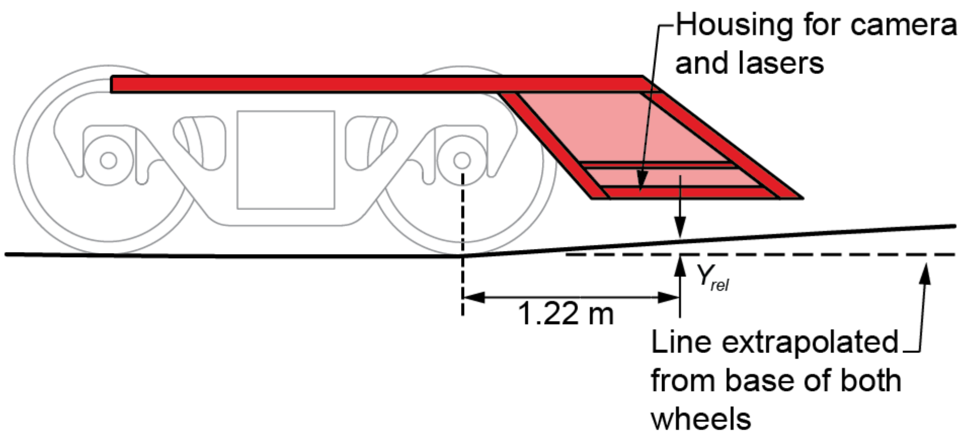

Figure B-4. Illustration of The University of Nebraska rolling deflection system used to assess the stiffness of the track

On October of 2013, University of Alberta in collaboration with CN used this system to evaluate the track stiffness along the LLBS. The average Yrel measure (see **Figure B-4**) was calculated and used as an index associated with track stiffness. **Figure B-5** shows the variation of this stiffness index along the LLBS. High stiffness index values correspond to high total deflection of the rail, and represent soft foundations. Both the track stiffness and surface roughness calculations were done for 0.1-mile long sections of the LLBS.

B.5. Track performance measurements

In Canada, Transport Canada has established track configuration standards called the Rules Respecting Track Safety (RRTS), commonly referred to as Track Safety Rules. Track irregularities that exceed these standards are considered track defects. The standard varies for each class of track and represents the minimum condition for a track to be considered safe. Therefore, a track defect requires prompt correction and operationally can be considered as a serviceability-related failure of the railway structure. As such, the performance of a railway track can be measured by the frequency of track defects recorded. The common measures used to define track defects are shown in **Figure B-6**.

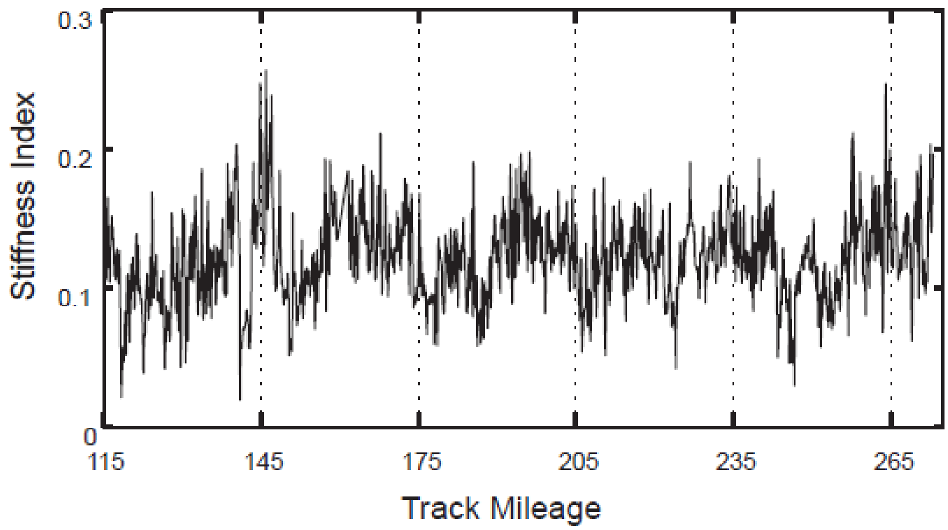

Figure B-5. Variation of the track stiffness index along the LLBS

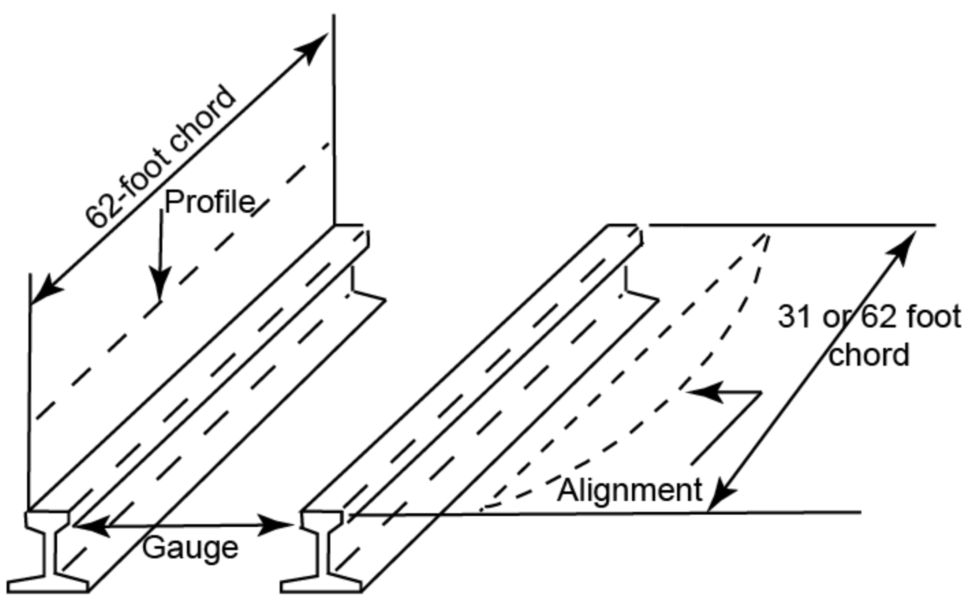

Figure B-6. Common measures used to define track defects. After (25)

Track defects are considered to be the main source of dynamic wheel-rail contact forces. An increase of these contact forces is associated with increases in the rate of track deterioration (4).

These defects are also the leading cause of train accidents (26). In the United States, 658 of the 1890 train accidents since 2009 were attributed to track irregularities (35%). These accidents resulted in $108.7 million in losses and damage (27). In Canada, according to Transportation Safety Board of Canada (www.tsb.gc.ca), about 14% of the main track incidents between 2004 and 2013 have been related to track defects. Because of its significance on the safety of operations, track defect frequency was selected as the track performance measure for further analysis. **Table B-1** shows the list of track defects considered in this study and their definitions. **Figure B-7** shows the distribution of track defects along the LLBS between 2010 and 2013.

Table B-1. Track defects used here and their definitions

Defect	**Description**
Crosslevel	Difference in elevation between the top surface of the rails at a point in a tangent track
Profile	Deviation of rail surface measured over a 62- foot chord
Alignment	Amount by which tangent track deviates from straight or spiral and curves deviate from design curvature
Warp	The difference in cross level between any two points less than 62 feet apart

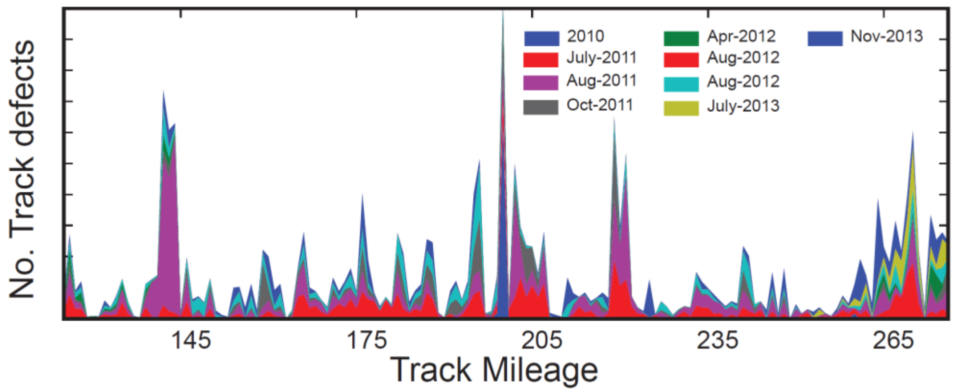

Figure B-7. Track defects along the LLBS, 2010 - 2013

B.6. Relationship between track quality and performance at the LLBS

The stiffness index (Y_{rel}) was compared against the number of track defects between 2010 and 2013 (**Figure B-8**). The comparison is for each of the 0.1-mile sections of the LLBS. The track was then divided into 10 different ranges in terms of stiffness values (**Figure B-9**). All ranges are equal in total track length (each range with a length equal to 10% of the LLBS, or 27.4 miles). Ranges have different level of stiffness index. In **Figure B-9**, the Range 1 represents the 10% of track that has the lowest deflection and therefore the highest stiffness. Alternatively, Range 10 is the 10% of track that has the highest deflection and therefore the lowest stiffness. Track stiffness decreases as we move from Range 1 to Range 10.

The track defects within each range were calculated combining the track defects in **Figure B-8** and the stiffness indexes that correspond to the limits of each range in **Figure B-9**. The results are presented in **Figure B-10**. This figure shows how the number of track defects within Range 10 (softest 10% of the LLBS) is significantly larger than those occurred within Range 1 (stiffest 10% of the LLBS). These results also show a tendency to find more track defects as the stiffness of the track decreases.

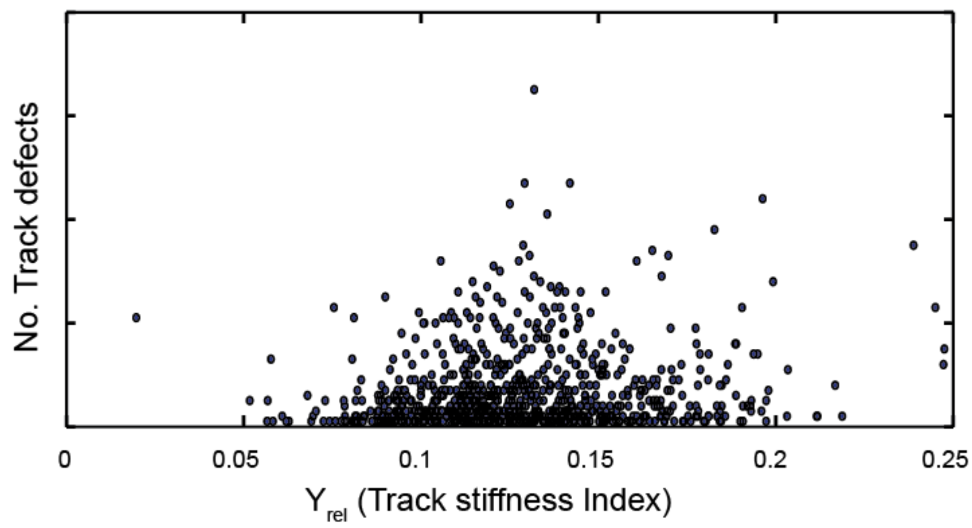

Figure B-8. Track defects and stiffness index

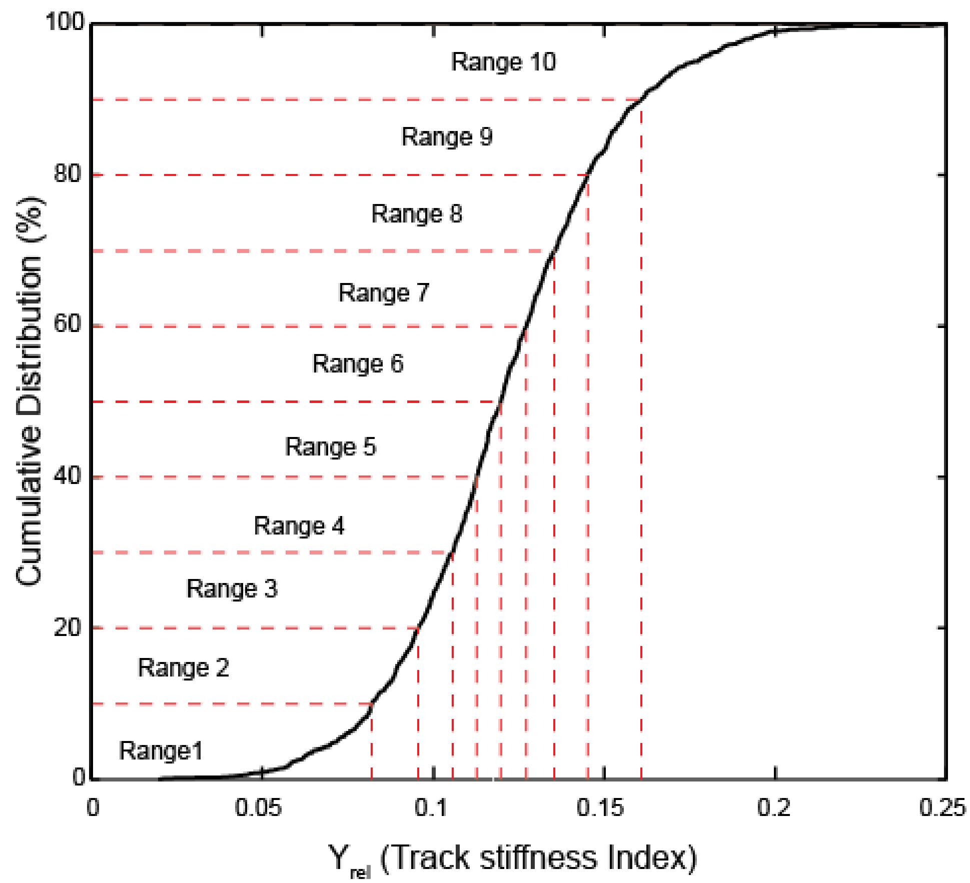

Figure B-9. Definition of different ranges of Track stiffness index

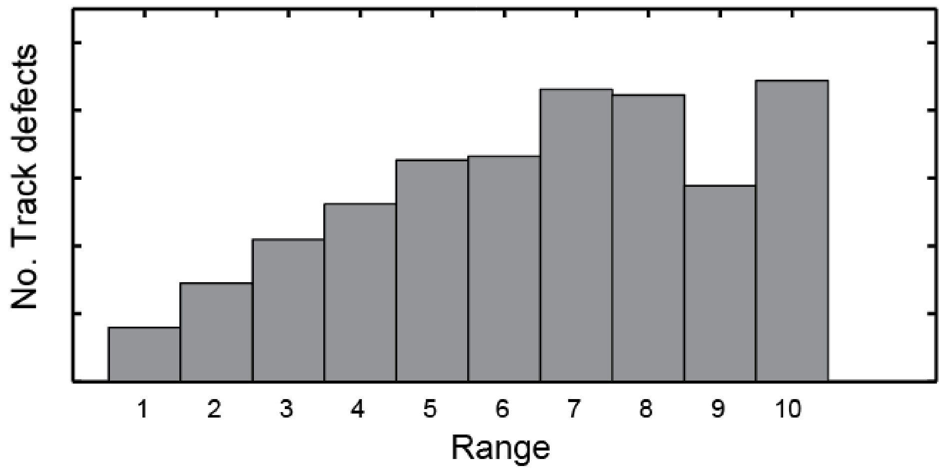

Figure B-10. Total number of track defects per stiffness range

The track surface roughness index was compared against number of track defects following the same approach discussed previously. The results of the analysis are presented in **Figure B-10**. This figure shows how the number of track defects increases with increasing track surface roughness.

B.7. Performance-based hazard chart for railway operations along the LLBS

The average number of track defects per year and per tenth-of-mile was calculated for each track stiffness range and track surface roughness range. Also, the 80th percentile of track defects within each range was calculated. The 80th percentile is the value larger than 80% of observations. **Figure B-12** shows the calculations for track stiffness and surface roughness along the LLBS.

Both statistical point estimates, the average and the 80th percentile, show the trends between track defect frequency and track quality measurements. While the average shows the trend of the average frequency of track defects, the 80% percentile can be adopted as the trend of extreme track defect frequencies for a given track quality Range.

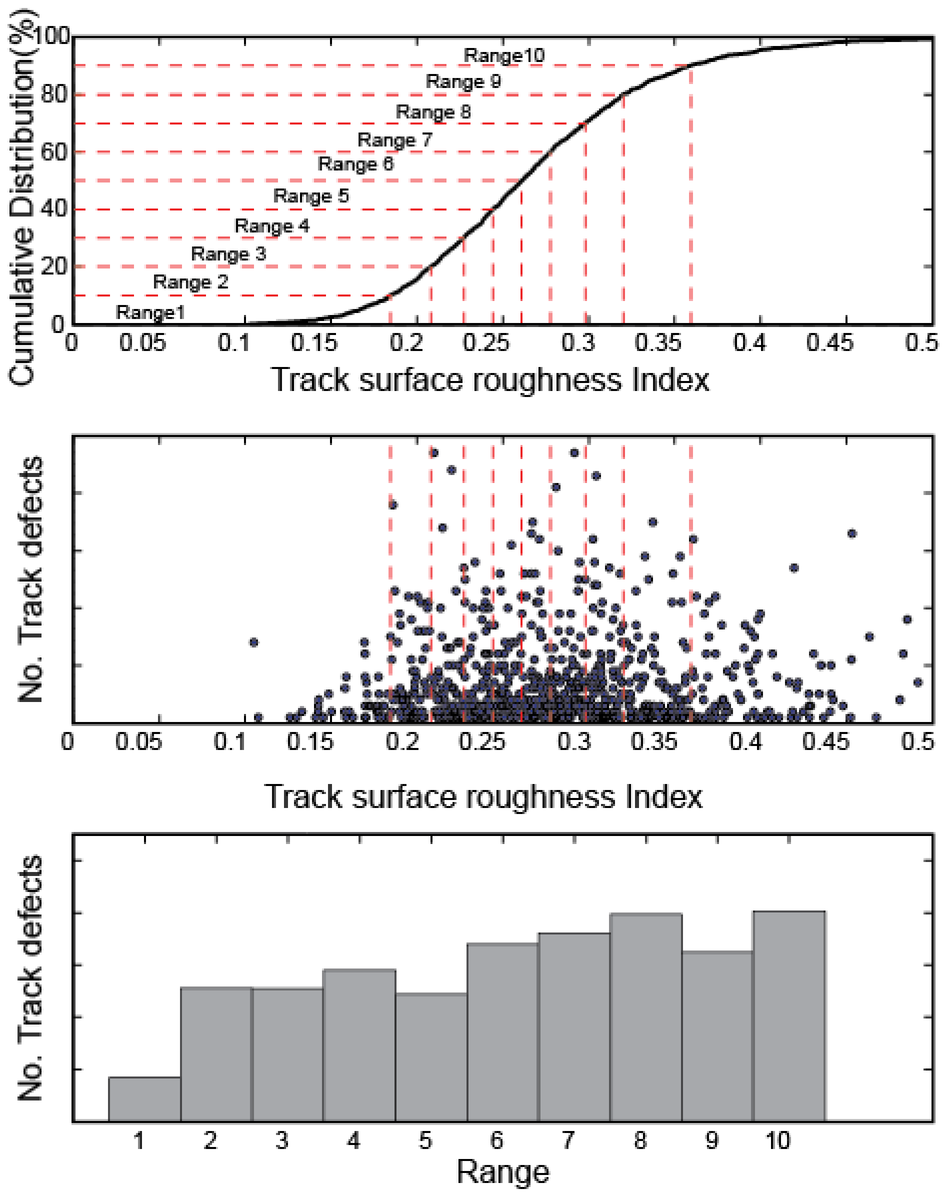

Figure B-11. Range of Track-surface-roughness index (a), comparison with defects (b), and track defects per range (c)

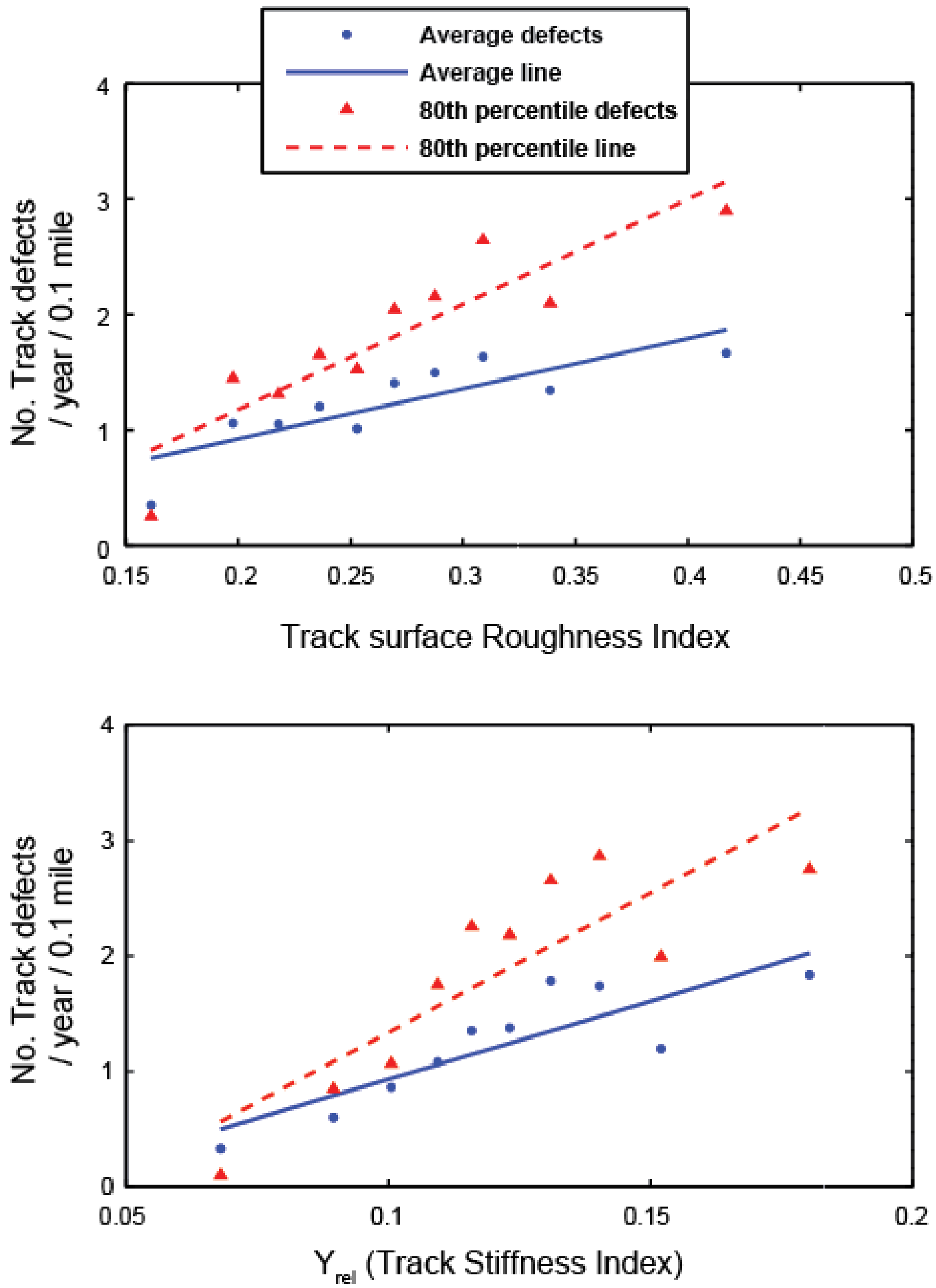

Figure B-12. Average and 80th percentile of track defect frequency compared with track surface roughness and stiffness in the LLBS

The 80th percentile best-fit-lines (red dashed lines in **Figure B-12**) are used for identification of sections susceptible to track defects. Defining thresholds for number of defects will depend on the required standard of care for the particular section. This is tied to the operator's goals regarding the fluidity and safety of the network. In this regard, it is the operator's responsibility to define such thresholds.

For illustrative purposes and considering the operation characteristics of the study area, sections with less than one track defect per year are considered within an acceptable level of performance. The track surface roughness and stiffness associated with 1 track defect per year at the 80th percentile line are considered to be upper bounds of smooth and stiff sections, respectively. Similarly, track surface roughness and stiffness associated with 2.5 track defects per year or more at the 80th percentile line are considered to be a concern. Sections above these values are classified as rough and soft sections, respectively. **Figure B-13** shows the threshold values adopted and the descriptors chosen for the degree of track roughness and stiffness.

The threshold values in Figure B-13 are used to develop a hazard chart. This chart considers all possible combinations of track surface roughness and stiffness (**Table B-2**). The 9 combinations were ranked in order of track quality. Here, low quality track is associated with sections where track defects are more likely to occur, and therefore are more hazardous. In this chart, the Ranking Level 1 corresponds to the higher relative hazard level. Sections within Ranking Level 1 would be priority for maintenance work allocation. Ranking Level 9 corresponds to the lower relative hazard level and sections within this level would be considered as the higher quality sections. This numeric ranking (last column in **Table B-2**) associated with each track quality description is based on the qualitative experience of the operator and the number of track defects presented in Figure B-10 and Figure B-11.

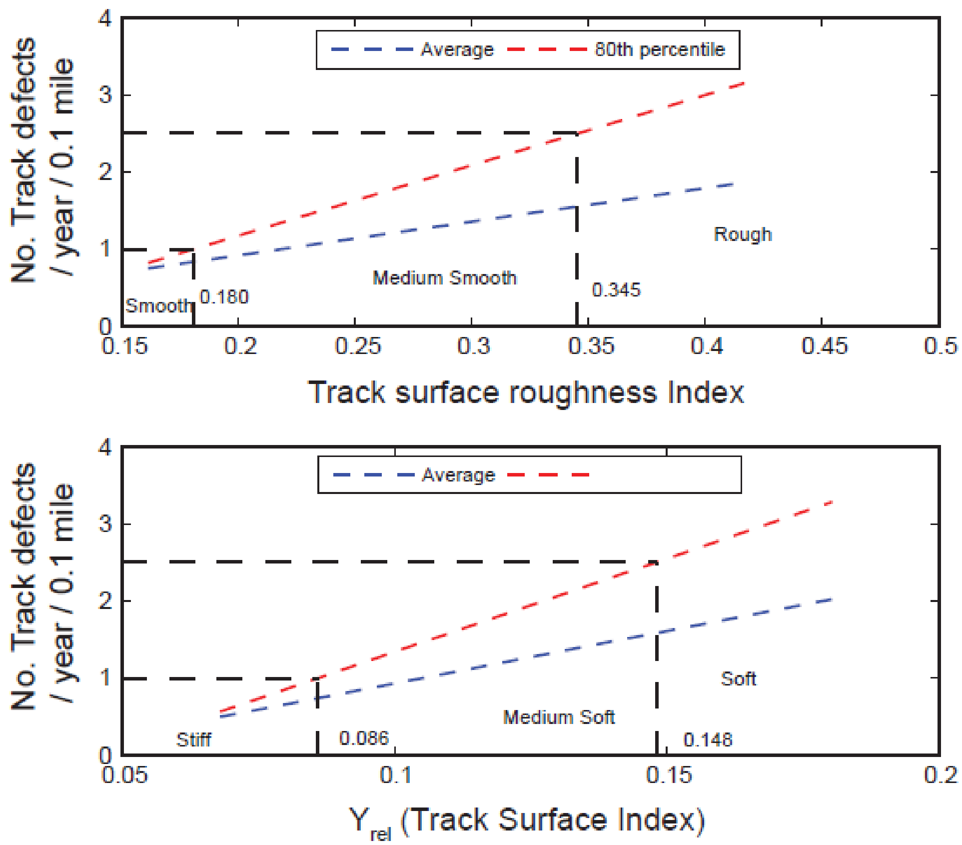

Figure B-13. Threshold values for track surface roughness and stiffness in the LLBS

B.8. Conclusion

Track quality measurements were combined with track performance measurements along a railway branch connecting Edmonton and Fort McMurray in Alberta, Canada. Track geometry and vertical deflection were used as track quality measurements related to track roughness and stiffness. The frequency of track defects was chosen as a measurement of track performance. The relationship between these quality and performance measurements was used to develop a semi-quantitative hazard chart. This chart is proposed as a potential tool for proactively allocate maintenance work and minimize the risks associated with track defects long the railroad track analyzed. The approach shows promising results, and it is now at a validation stage. The authors caution the reader that this is a tool under development and that, at this stage; the chart requires to be developed at each particular subdivision based on its quality and performance records. The hazard chart presented here ranks the hazard level associated with track quality based on the operator's experience and number of defects for each quality descriptor. Further research is ongoing to incorporate statistical analyses of operation delays attributed to rail defects and adopt a numerical ranking level based on a fully quantitative approach.

Table B-2. Hazard chart for track defects along the LLBS

Track Stiffness	**Track surface Roughness (TSR)**	**Description**	**Ranking Level**
Yrel<0.10	TSR<0.217	Stiff and smooth track	9
	0.217<TSR<0.327	stiff and medium rough track	8
	0.3267<TSR	Stiff and rough	6
0.10<Yrel<0.141	TSR<0.217	medium stiff and smooth track	7
	0.217<TSR<0.327	medium stiff and medium rough	4
	0.3267<TSR	medium stiff and rough track	3
0.141<Yrel	TSR<0.217	soft and smooth track	5
	0.217<TSR<0.327	soft and medium rough	2
	0.327<TSR	soft and rough track	1

B.9. Acknowledgements

The authors would like to acknowledge the Canadian National Railway (CN) for facilitating the necessary information for analysis. Special thanks in particular to Tom Edwards, Melissa Ruel and Mario Ruel from CN. This research was made possible through the Canadian Rail Research Laboratory (CaRRL) (www.carrl.ca), which is funded by the Natural Sciences and Engineering Research Council of Canada (NSERC), Canadian Pacific Railway, Canadian National Railway, the Association of American Railways – Transportation Technology Centre Inc., the National Research Council of Canada, Transport Canada and Alberta Innovates – Technology Futures.

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