Evolution of Truck Weight Limits and Bridge Live Load Models: Evidence of Impacts from Manitoba, Canada

By

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Abstract

This research presents evidence of the impacts of truck size and weight regulation changes on bridge live load models, truck operating gross vehicle weights and bridge structure reliability, specifically focusing on the context of Manitoba, Canada. This work addresses previous research gaps and contributes knowledge to this topic by: (1) examining historical relationships between truck size and weight regulations and bridge design codes; (2) conducting a retrospective, longitudinal study using a unique time-series of truck weight data to provide insights about the magnitude and timing of the impacts of truck weight regulatory changes on truck operating gross vehicle weights; and (3) evaluating the reliability of typical bridge types designed according to modified live load models and subject to differing truck live loads. Key findings from this work include several insights and observations. First, while at times bridge codes are released in conjunction with expected regulation changes, there is often a delay in the issuance of revised design codes, thus highlighting the importance of assessing current changes to the truck fleet for future bridge code live load models. Second, truck carriers take advantage of increased truck weight limits; however, the truck configuration and type of commodity influences the sensitivity to the weight limit changes. Similarly, the timing of the response is dependent on the ease of implementation and immediate increases are observed for weigh-out commodities when fleet modifications are not required. Third, three bridge types designed to the previously unanalyzed live load model specified in Manitoba are found to maintain a level of safety for the girders that exceeds target code requirements with reliability indices ranging from 4.65 to 5.04, depending on the considered live load statistics. This indicates a potential capacity for the considered structure types to safely withstand increased traffic load effects. By studying specific topics influenced by truck size and weight regulation changes and providing evidence of the impacts on truck operating weights and bridge infrastructure, this thesis contributes new insights to address gaps in previous research. It highlights the importance of monitoring changing truck weights and configurations to maintain the level of safety of bridge infrastructure.

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Glossary of Acronyms and Abbreviations

ADTT	Average Daily Truck Traffic
AASHTO	American Association of State Highway and Transportation Officials
FHWA	Federal Highway Administration
GVW	Gross Vehicle Weight
LCV	Longer Combination Vehicle
MHTIS	Manitoba Highway Traffic Information System
MoU	Memorandum of Understanding
NCHRP	National Cooperative Highway Research Program
RTAC	Roads and Transportation Association of Canada
TRB	Transportation Research Board
TSW	Truck Size and Weight
VDT	Vehicle Distance Travelled
WCHSP	Western Canada Highway Strengthening Program
WIM	Weigh-in-Motion
3-S2	Five-axle tractor semitrailer
3-S3	Six-axle tractor semitrailer
3-S3-S2	Eight-axle B-double
3-S2-4	Turnpike double

1.0 Introduction

1.1 PURPOSE

It is conceptually well understood that bridges must be designed to support vehicular live loads (in addition to other loads). Bridge design codes simplify the wide range of truck loadings into a single notional load that envelopes the potential live load effects over the lifespan of the structure, while limiting the maximum probability of failure to within acceptable thresholds. During bridge code development, it is often assumed that the legal loads, and subsequent truck operating weights, will stay constant over the 75-year bridge lifespan (Kulicki et al., 2007). Although this assumption eliminates the need to predict future changes in truck operating weights—including those that may be instigated by changes in truck size and weight (TSW) regulations, it implies that bridge capacity could become a barrier if such increases were to occur.

When modifications to TSW limits are found to be feasible, from a bridge perspective and otherwise, and are in-turn implemented, there is uncertainty about how truck operating weights will correspondingly change. A level of understanding of the implications of policy changes is important for guiding practitioners in their predictions of how the new TSW limits will be adopted by trucking operators, and when.

Canadian provinces have implemented several changes to the TSW regulatory environment over the past five decades. These changes allow for a unique, retrospective assessment of the evolution of TSW limits, truck operating weights, and bridge live load models and reliability. The purpose of this thesis is to examine this historically complex relationship, while providing evidence of the impacts of changes in truck weight limits and bridge live load models within the context of Manitoba, Canada.

1.2 BACKGROUND AND NEED

The regulation of truck size and weight is complex and influences many facets of road freight transportation, including truck productivity, market competitiveness, energy and emissions, infrastructure design and maintenance, and road safety and system performance. Despite these widespread influences, this thesis focuses on the impact of TSW regulations on bridge infrastructure, an issue which has been recognized as a primary economic barrier to change by several previous TSW studies (National Academies of Sciences, Engineering and Medicine, 2003; National Academies of Sciences, Engineering, and Medicine, 2019; U.S. Department of Transportation, 2015). To consider the effects of TSW regulations on bridges, it is critical to understand their impact on truck operating weights and on bridge design codes. This section addresses these two issues.

1.2.1 Truck Size and Weight Regulations

Truck size and weight regulations define the maximum weight on any axle (or axle group); the maximum weight of the entire truck (gross vehicle weight or GVW); various dimension limits, for example the length, width and height of the truck/trailer; and the required spacing ranges within an axle group and between axle groups (i.e., interaxle spacing) for various allowable truck configurations. In Canada, these limits are enforced provincially primarily through the use of truck weigh scales, both permanent and portable. Enforcement of TSW limits is important for protecting infrastructure, reducing wear on highway pavements and shoulders, and limiting impacts on bridges due to illegally overloaded vehicles. Much like any high-level policy decision, monitoring the changes in operating trucks following TSW regulation modifications is also important for ensuring that unintended and undesirable consequences do not result (Woodrooffe et al., 2010).

Changes in TSW regulations have often been driven by external forces as a consequence of an expanding and improving highway system, advances in vehicle technology and bridge design methods, and economic pressures to reduce freight shipment costs (Transportation Research Board 2002). As a result, decisions regarding TSW regulations are both political and technical, acknowledging that some parties may be benefitted at the expense of others. Ideally, TSW regulations should be optimized to balance the potential costs of heavier, longer trucks with the expected productivity benefits for freight transportation to produce the greatest public benefit from the highway transportation system (Woodrooffe et al., 2010).

Canadian agencies have focused on the development of TSW regulations that facilitate safe and efficient freight movement, while simultaneously limiting the consumption of public infrastructure assets. In Canada, TSW regulations are under the jurisdiction of provinces. Historically, there was little collaboration between provinces, resulting in a wide range of allowable TSW limits across Canada in the early part of the 20th century (Woodrooffe et al., 2010). However, as interprovincial truck transportation developed along with the highway system in the 1960s and 1970s, these varying weight regulations limited cross-country transport (Woodrooffe et al., 2010).

This productivity barrier initiated detailed technical studies and national discussions to harmonize the regulations interprovincially and improve the overall efficiency of truck transportation without causing significant negative impacts to road safety and infrastructure (Roads and Transportation Association of Canada, 1980; Roads and Transportation Association of Canada, 1986; Woodrooffe et al., 2010). These studies resulted in several TSW policy changes over the past five decades, most notably being the nationally signed RTAC Memorandum of Understanding in 1988. According to Woodrooffe et al., these policy changes have increased the uniformity of truck configurations operating nationally, improved national trucking productivity, yielded positive economic impacts and encouraged use of vehicle configurations with superior dynamic performance (2010).

The United States has expressed similar interest in developing a uniform set of TSW limits for use on the interstate system and has completed federal studies and reviewed several proposal evaluations to explore this (National Academies of Sciences, Engineering, and Medicine, 1990; National Academies of Sciences, Engineering, and Medicine, 2003; Transportation Research Board, 1986; Transportation Research Board, 1989; Transportation Research Board, 1990; Transportation Research Board, 2002; U.S. Department of Transportation, 2000; U.S. Department of Transportation, 2015). Despite this, limitations in understanding and predicting the changes in the operating trucks, and the resulting costs and benefits of the new truck fleet, have limited policy change. Additional studies have been recently recommended to fill in gaps and uncertainties in the previous works and better estimate the effects of modified TSW limits on safety, compliance, modal shifts, pavements and bridges prior to implementation (National Academies of Sciences, Engineering, and Medicine, 2019). Thus, the Canadian context provides unique insight for understanding key outcomes of several TSW limit changes that have occurred over the past five decades.

1.2.2 Impact on Truck Operating Weights

To assess the impacts of TSW regulation changes on aspects such as emissions, safety, pavements and bridges, the operating weights of trucks must first be reasonably understood. Freight carriers will respond to policy changes, possibly by directly taking advantage of the allowed weight benefits with their existing fleet or by modifying their fleet to new, more efficient truck configurations (Clayton and Lai, 1986; Regehr et al., 2009a). The truck configuration and the commodity hauled may also influence this decision (Clayton and Nix, 1986; Nix et al., 1989). Predicting the trucking industry response is an important consideration for estimating the costs and benefits of implementation. Similarly, understanding when size and weight advantages may be capitalized on by carriers is also essential for agencies, especially when predicting infrastructure deterioration and estimating maintenance budgetary requirements (Regehr et al., 2009a; U.S. Department of Transportation, 2015).

Typically, TSW studies attempt to estimate future truck activity by: developing models that characterize expected freight demands across the network, conducting a modal diversion analysis to meet those demands, predicting a distribution of operating weights to the prospective truck configurations, and generating estimates of vehicle distance travelled (VDT) by truck configuration (U.S. Department of Transportation, 2000; U.S. Department of Transportation, 2015). These estimates are critical because most subsequent impact assessments, including safety, pavement, emissions, bridges and compliance, use VDT estimates as input data (U.S. Department of Transportation, 2000; U.S. Department of Transportation, 2015). Validating the reliability and accuracy of these inputs is important but has been challenging historically and the uncertainties of these analysis methods have precluded policy actions at times (Transportation Research Board, 2002).

As discussed in Chapter 3, a longitudinal study of the Canadian experience provides a unique opportunity to analyze the impact of historic TSW regulatory changes on the truck fleet mix and operating gross vehicle weights, and helps address questions surrounding the time that it takes for the trucking industry to respond to regulatory change. Information resulting from case studies such as the one conducted in Manitoba, may provide a means for validating models that have previously been developed and also generates transferable insights to other jurisdictions.

1.2.3 Impact on Bridge Live Load Models

Inherent to any change in operating trucks, the effects on infrastructure, particularly bridges, is a key concern. Despite the innate and intuitive relationship between trucks, bridges and bridge live load models, this relationship has been historically complex.

The most basic impact of TSW regulations on bridges is related to the truck operating weights and the available capacity of bridges to carry any increased truck loading, while maintaining a minimum accepted level of safety. This is especially important when considering that a large proportion of bridge infrastructure in North America is approaching the end of its initial design life (assuming that rehabilitations have not been completed to extend the structure life), and noting that many of these were constructed to different bridge design codes and live load models over the past century. For example, in the Manitoba Infrastructure bridge inventory, 37% of the structures are known to be more than 50 years old and this proportion increases to 48% when including structures with unspecified construction dates (A. Pankratz, personal correspondence, 2020). Similarly, in the United States, 41% of all bridge structures are more than 50 years old and only 45% of the national bridge inventory is classified as being in Good Condition (U.S Department of Transportation, 2020).

Bridge costs associated with increasing allowable truck weights consider accelerated maintenance costs as well as strengthening or replacement costs required to maintain bridge level of service over a typical 75-year design lifespan (National Academies of Sciences, Engineering, and Medicine, 2003). Previous studies have estimated that the cost to strengthen bridges with insufficient capacity would produce the most significant cost following increased allowable GVW limits (National Academies of Sciences, Engineering, and Medicine 2019). Predicting how modified vehicle loads will affect existing bridge deterioration and impact the remaining service life becomes complex when considering the various bridge types, the vintage of those bridges and the bridge design code that was applicable at the time of construction.

Trucks travel on highways producing a wide range of load effects resulting from the various truck configurations and weights. The envelope of the load effects must be approximated and simplified into a loading model for the design and evaluation of bridges. Factors of safety and projections of maximum expected live load effects over the design life of the bridge are included in the bridge code to account for these variabilities, although it is typically assumed during code development that the allowable truck weights will remain constant over the lifespan of the bridge (Kulicki et al., 2007). The live load models specified in the design code are calibrated with load and resistance factors and reliability analyses are conducted to ensure a uniform and consistent level of safety, or probability of failure, for bridges designed according to the code, regardless of bridge type and span length (Nowak, 1995). Agencies can conduct reviews and evaluations of their current truck fleet and may modify the live load specifications accordingly, especially when considering specific geographic regions that may regularly carry special permit loads. As a result, TSW regulations and operating traffic load effects have an impact on the development and calibration of bridge design codes and their live load models, and the two have evolved in concert.

As discussed previously, bridge condition and capacity are key considerations for cost assessments related to TSW changes and future allowable truck weight increases may be governed by the levels of excess bridge capacity available. Acknowledging this, several provinces in Canada have developed modified bridge design live load and evaluation models that exceed the minimum requirements specified in the national bridge design code (Alberta Transportation, 2018; British Columbia Ministry of Transportation and Infrastructure, 2016; CSA, 2019). Such increases allow added capacity for industry specific permit trucks or other overloaded vehicles that may cross highway bridge infrastructure, while also providing the opportunity for further increases in legal weight limits if desired (UMA Engineering Ltd., 2001). However, modification of the live load models specified by the bridge code for design and evaluation requires assessments to determine if the bridge code load and resistance factors remain adequately calibrated for the different structure types and span lengths.

Overall, bridges are affected by TSW regulations in several ways. Most intuitively through the changing loading conditions, due to new truck configurations and load allowances that a structure must safely carry, but also by potential modifications to the bridge design code and associated live load models, which are referenced for the design and evaluation of bridge structures.

1.2.4 Themes, Connecting Concepts and Research Gaps

Despite the previous research surrounding TSW regulatory changes and corresponding impact assessments, the complex and interconnected relationships between policy, truck operating weights, and bridge code and reliability have not been well understood historically. By taking a multi-paper approach, this thesis aims to explore specific aspects related to this overall theme and address key knowledge gaps.

First, the evolution of TSW regulations and bridge design code live load models over the past five decades is documented. Although the relationship between legal truck weight limits and bridge code live load models appears self-evident, it has been historically hard to establish due to limited collaboration between truck size and weight policy makers and bridge code authors (Csagoly and Dorton, 1978).

Second, the impacts of TSW regulation changes on truck operating gross vehicle weights is evaluated by examining a longitudinal set of truck weight data collected in Manitoba. This unique truck weight dataset was recorded over a forty-five year period during which several TSW regulation changes occurred, allowing for direct observations and insights about the magnitude and timing of changes to truck GVW distributions. The classification of trucks in the weigh scale data also allows for specific commentary by truck configuration. These retrospective insights can be transferrable to other jurisdictions hoping to understand the impacts of TSW limit modifications. Despite the availability and use of predictive models for estimating the truck operating weight response by configuration, difficulty in validating these models has limited this approach, leading to uncertainty in understanding the effects following proposed TSW changes and precluding action at times.

Finally, although nationally agreed upon regulations specify the maximum allowable TSW limits on major highways in Canada, individual provinces can choose to allow heavier vehicles to operate on their highway network, typically in response to industry specific live load requirements. To account for this, localized modifications to the bridge design code live load model also follow to ensure that bridge level of safety is maintained. A case study approach is used to determine the reliability of three typical bridge types in Manitoba subject to observed truck operating weights, leveraged from the historical truck weight dataset, and designed in accordance with a local live load model specified by Manitoba Infrastructure, the HSS-25. Despite satisfactory bridge performance over the past five decades, the level of safety of bridges designed according to the HSS-25 model has not been formally evaluated to date.

Each paper reveals insights about the complex relationships between truck size and weight regulatory changes, bridge live load models, truck operating gross vehicle weights and bridge structure reliability, while presenting evidence of the impacts of these relationships using data collected in Manitoba, Canada.

1.3 OBJECTIVES AND SCOPE

Given the current knowledge gaps and uncertainties, three primary research questions form the objectives of this research:

1. How have bridge live load models evolved in concert with truck weight limits?
2. How have truck gross vehicle weights evolved as a function of truck weight limits?
3. How has bridge reliability been impacted by changes in live load models and truck operating weights in Manitoba?

In addressing these questions, the intent is to clarify concepts and considerations resulting from TSW regulation changes focusing specifically on the impacts on trucks and bridges. The thesis is constrained as follows:

- Spatially, the analysis focuses on the case of Manitoba, Canada, with data collected along the east-west Trans-Canada Highway in the province. Consequently, certain results pertain specifically to Manitoba (e.g., the bridge reliability analysis using the Manitoba Infrastructure live load model). However, many of the insights and methods are considered transferrable to other jurisdictions seeking to improve understanding of the impacts of TSW regulation changes.
- It is recognized that some North American bridge design code development occurred prior to the 1970s; however, the discussion concerning the evolution of bridge design codes focuses on the advancements in load and resistance factor design methods, which occurred during the selected five-decade time period. Additionally, the discussion is limited to changes in the live load model and does not provide a summary of changes to other aspects of the bridge design code, such as load factors, load combinations and the resistance model.
- The truck GVW data used for the retrospective, longitudinal study was available for a forty-five-year period and commentary focused on observations and insights within that time frame. Limited data collection prior to the 1974 TSW regulation change precluded detailed discussions regarding the impacts of that particular policy modification.
- The technical scope of the impacts of TSW regulation change on truck operating weights focuses on changes in the truck GVW distributions and does not cover in detail fleet mix changes or other aspects related to freight productivity and costs, as has been previously documented elsewhere.
- The observed truck data referenced for the bridge reliability analysis was limited to data collected over a two-decade period. Observed truck weight data prior to

this time did not include axle weight distributions or spacings and therefore was insufficient for the determination of live load effects on a structure.

- The bridge reliability analyses were carried out using a case-study approach for three prominent bridge types in Manitoba, considering simply supported structures with a specified span length range of 16 to 26 m. The structure types and span lengths were selected based on available bridge inventory data provided by Manitoba Infrastructure and extensions of the results to other bridge types and span lengths was beyond the scope of the work.

Despite the identified constraints, conclusions from this research may stimulate a further assessment of potential lessons learned from Canada and transferability of findings following TSW regulation modifications over the past five decades.

1.4 APPROACH

The objectives of this research were met by individually examining key elements that are impacted by TSW regulation changes, namely bridge live load models, truck operating weights and bridge reliability. The research approach comprises the following five points:

1. A historical examination of bridge design codes and truck size and weight regulation changes applicable in Canada over the past 50 years, with particular focus on the Manitoba context.

Previous Canadian and American bridge design codes and commentaries, and other available literature, was referenced to understand the bridge design code changes that occurred over the past five decades. Journal publications and technical studies were reviewed to help determine the influence of historic policy changes on bridge design code modifications.

2. A retrospective evaluation of the impacts of truck size and weight regulation changes on truck operating gross vehicle weights.

Using the historical examination of TSW regulations as the foundation for this work, forty-five years of truck weigh scale datasets were referenced to conduct a retrospective evaluation of changes in truck operating GVW distributions for prominent vehicle configurations on the Trans-Canada highway in Manitoba.

3. Statistical analyses to establish the significance of changes in the observed truck weight distributions.

Two-sample Kolmogorov-Smirnov tests were used to compare the changes in distributions over time and determine whether the differences were significant enough to reject the null hypothesis that both samples came from a population with the same distribution.

4. Selection of case study bridge types for a reliability analysis.

The Manitoba Infrastructure bridge inventory was referenced to select three typical bridge types and span length ranges for the analysis. The most prominent structure types on provincial trunk highways were considered. Using representative drawings sets for the selected bridge types, as provided by Manitoba Infrastructure, the structural resistances were estimated based on the bridge geometry and cross-sectional properties.

5. An analysis of bridge reliability using historical truck weight records to determine the level of safety of bridges designed according to a modified live load model.

Two decades of truck weigh scale data and weigh-in-motion data were used to determine the maximum daily Manitoba-specific live load moment and shear effects, which were then projected to estimate the maximum load effects over the bridge lifespan. The reliability indices of the structure types over a variety of span lengths were calculated to provide commentary on the level of safety, compared to the target level of safety specified in the bridge design code.

The results are presented as a sandwich thesis. The following chapters include research papers that provide additional details regarding the specific methodological approaches used to meet the objectives of this research. Each paper leverages historical information to describe evidence of the impacts of TSW limit changes.

1.5 THESIS ORGANIZATION

This thesis comprises 5 chapters, including this introductory chapter. The three chapters following the introduction are self-contained research articles that meet the objectives of the research specified in Chapter 1. Chapter 2 explains the evolution of TSW regulations and bridge design code live load models. Chapter 3 provides a longitudinal study on the impacts of TSW regulations on truck operating GVWs in Manitoba. Chapter 4 analyzes the reliability of three bridge types subjected to observed truck weights in Manitoba and designed with a local uncalibrated live load model. The conclusion in Chapter 5 provides a discussion of the findings and summarizes the contribution of the research towards improving the understanding of the impact of TSW regulation changes on bridge live load models, truck operating GVWs and bridge structure reliability, specifically within the context of Manitoba, Canada. Recommendations for future research are also provided.

1.6 TERMINOLOGY

Average Daily Truck Traffic (ADTT) – the number of trucks passing a point on a stretch of road during a day, counted for more than one day but less than one year, divided by the number of days in that time period.

Axle Group – a defined group of adjacent axles within a single vehicle unit that equalizes loads on adjacent axles within 1000 kg. Includes steering axles, single axles, tandem axles and tridem axles (Council of Ministers of Transportation and Highway Safety, 2019).

Axle Load – the sum of all tire loads of the wheels on an axle; the total weight transmitted to the highway by the axle or axle group; a portion of the gross vehicle weight (Council of Ministers of Transportation and Highway Safety, 2019).

Axle Spread – the longitudinal distance between the extreme axle centers of the axle group (Council of Ministers of Transportation and Highway Safety, 2019).

B-Double – a vehicle composed of a tractor, a semitrailer, followed by another semitrailer attached to the first semitrailer by the means of a fifth wheel mounted on the rear of the first semitrailer (Council of Ministers of Transportation and Highway Safety, 2019).

Bridge – a structure that provides a roadway or walkway for the passage of vehicles, pedestrians or cyclists across an obstruction, gap or facility and is greater than 3 meters in span (CSA, 2019).

Dynamic Load Allowance – an equivalent static load that is expressed as a fraction of the traffic load and is considered to be equivalent to the dynamic and vibratory effects of the interaction of the moving vehicle and the bridge, including the vehicle response to irregularity in the riding surface (CSA, 2019).

Gross Vehicle Weight (GVW) – the total weight transmitted to the highway by a vehicle or combination of vehicles (Council of Ministers of Transportation and Highway Safety, 2019).

Interaxle Spacing – the longitudinal distance separating two axles or axle groups calculated from the centers of the two adjacent axles (Council of Ministers of Transportation and Highway Safety, 2019).

Limit States – those conditions beyond which a structure or component ceases to meet the criteria for which it was designed; typically referring to ultimate, serviceability and fatigue (CSA, 2019).

Load Effect – any effect on or response of a structural component due to loads, forces, imposed deformations, or volumetric changes (CSA, 2019).

Load Factor – a factor applied to loads to take into account variability of loads, lack of precision in analysis for load effects, and reduced probability of loads from different sources acting simultaneously (CSA, 2019).

Reliability Analysis – in the context of bridges, a method to assess the level of safety (or probability of failure) with respect to a given limit state.

Reliability-Based Code Calibration – a process to determine load and resistance factors so that bridges designed and evaluated according to a bridge code will achieve a specified level of safety.

Resistance Factor – a factor applied to the unfactored resistance of a component or material at the ultimate limit state to take into account the variability of material properties, quality of work, type of failure and uncertainty in the prediction of the resistance (CSA, 2019).

Semitrailer – a vehicle that is designed to be towed by another vehicle and is so designed and used that a substantial part of its weight and load rests on or is carried by the other vehicle or a trailer converter dolly through a fifth wheel and kingpin combination (Council of Ministers of Transportation and Highway Safety, 2019).

Single Axle – One or more axles whose centers are included between two parallel transverse vertical planes less than 1.2 meters apart (Council of Ministers of Transportation and Highway Safety, 2019).

Span – longitudinal distance between the centerline of supports or bearing units of a bridge.

Steering Axle – the articulated lead axle or axles of a motor vehicle which govern the direction travelled by the vehicle (Council of Ministers of Transportation and Highway Safety, 2019).

Tandem Axle – an axle group containing two consecutive axles whose centers are not less than 1.2 meters apart and are attached to the vehicle in a manner which achieves equalized loading between the axles (Council of Ministers of Transportation and Highway Safety, 2019).

Target Reliability Index – the required level of safety that must be provided by the structural capacity of a bridge component; depends on the life safety and economic consequences of failure of a bridge component (CHBDC Calibration Subcommittee, 2002).

Tractor – a motor vehicle designed to and normally used to pull a semitrailer or a semitrailer and a full trailer or a semitrailer and a semitrailer (Council of Ministers of Transportation and Highway Safety, 2019).

Tridem Axle – an axle group containing three consecutive axles whose extreme centres are not less than 2.4 meters apart, are equally spaced and are attached to the vehicle in a manner which achieves equalized loading among the three axles (Council of Ministers of Transportation and Highway Safety, 2019).

Truck – a motor vehicle, other than a bus, that is either permanently fitted with a special purpose device, or is designed to and normally used to carry a load, that may operate as a single unit or may pull a trailer other than a semitrailer (Council of Ministers of Transportation and Highway Safety, 2019).

Weigh-in-Motion (WIM) – the process of estimating a vehicle's gross vehicle weight and the weights of individual axles or axle groups by measurement and analysis of dynamic tire forces.

2.0 **Evolution of Bridge Live Load Models and Truck Weight Limits: The Case of Manitoba, Canada**

Although the influence of legal truck weight limits on bridge code live load models appears self-evident, understanding that TSW limits effect the operating weight of vehicles that must be carried by bridge infrastructure, this relationship has been historically hard to establish due to limited collaboration between policy makers and bridge code authors. This technical note presents the evolution of bridge live load models and how they have adapted and responded to TSW regulation changes over the past five decades, specifically focusing on TSW regulation changes and bridge codes applicable in Manitoba, Canada. Through this, the chapter seeks to answer the first research question: How have bridge live load models evolved in concert with truck weight limits?

The material in this chapter was submitted for publication in the Canadian Journal of Civil Engineering on September 2, 2020 and is printed here with permission of co-authors Jonathan Regehr, Aftab Mufti, Basheer Algohi and Graziano Fiorillo. The paper is self-contained in the chapter with its own abstract, introduction, conclusion and references. The author of this thesis was the lead author and researcher and had principal responsibility for all aspects of the paper, while co-authors provided advice and reviews.

2.1 ABSTRACT

Truck size and weight regulations have been a key instrument used to improve trucking productivity, safety, and operational performance in Canada. In response to these changes, bridge design codes undergo modifications to envelope the potential range of trucks in operation. A five-decade timeline is presented: (1) to document how bridge codes and their live load models have evolved, with a focus on the Manitoba-specific HSS-25 truck, and (2) to discuss how responsive bridge design codes have historically been to changes in truck size and weight regulations. While at times bridge codes are released in conjunction with expected regulation changes, there is often delay in the issuance of revised bridge design and evaluation codes. Assessments of the current

truck fleet, which now includes long combination vehicles (LCVs), may be a consideration for future bridge design live load models.

2.2 INTRODUCTION

The trucking industry in Canada has evolved over the past several decades, responding to growth and changes of society and the need for improved productivity, better safety and operational performance, and more stringent emissions control. Since the 1970s, changes in truck size and weight (mass) limits have been a principal regulatory instrument used in Canada (and elsewhere) to achieve these objectives. In Canada, three primary regulatory changes have resulted in increases to the allowable size and weight of trucks over the past five decades: (1) federally-sponsored highway strengthening programs initiated in the 1970s, (2) the 1988 Roads and Transportation Association of Canada Memorandum of Understanding on Interprovincial Weights and Dimensions (RTAC MoU), and (3) regional policies regarding special permitting of longer combination vehicles (LCVs) (Regehr et al. 2009). These regulatory limits not only control the types and operating weights of truck configurations on highways, but also influence live load design parameters for current and future bridge infrastructure.

North American highway bridge design codes have evolved in recognition of the need to develop live load models that envelope the range of trucks operating on the highway network, several of which are depicted in Table 1. While this relationship appears self-evident, it has historically been challenging for jurisdictions to establish, especially because bridge code authors are typically not directly involved in administering truck size and weight limits (Csagoly and Dorton 1978). From the 1970s to 1990s, research into highway bridge design code development and calibration was ongoing in North America, as historic codes such as the AASHTO *Standard Specifications for Highway Bridges* looked to implement load and resistance factor design (LRFD) methods. The Province of Ontario led these efforts through their development of load and resistance parameter statistical databases and subsequent publication of the Ontario Highway Bridge Design Code (OHBDC) in 1979. Leveraging this work and other research conducted during this time, AASHTO also developed and calibrated a LRFD bridge code, with the first edition released in 1994. Shortly after this in 2000, the first edition of the Canadian Highway Bridge Design Code (CHBDC) was published, merging OHBDC with the previous Canadian CSA-S6 Code.

Table 2-1: Major Articulated Truck Configuration Groups

Configuration group	Representative truck configuration	Axle configurations
Three-, four-, and five-axle tractor semitrailers		2-S1, 2-S2, 3-S1, 3-S2 (shown)
Six-axle tractor semitrailers		3-S3
Eight-axle B-double		3-S3-S2
Specially permitted LCVs		3-S2-4 turnpike double (shown), others

The province of Manitoba, like its western provincial and territorial counterparts, has experienced several primary truck size and weight policy changes: (1) the 1974 Western Canadian Highway Strengthening Program (WCHSP) and its derivatives, (2) the 1988 RTAC MoU, (3) and the initiation and subsequent expansion of LCV permitting (Regehr et al. 2009; Wood and Regehr 2017). In addition to these primary changes, the province has implemented modifications to the allowable gross vehicle weight (GVW) on specified portions of its highway network. From a bridge perspective, the province of Manitoba is unique in the sense that it is influenced by both the AASHTO design code and CHBDC, as Manitoba Infrastructure (MI) requires new provincial highway bridge designs to be completed in accordance with AASHTO (with modifications), and the City of Winnipeg, like most other Canadian jurisdictions, designs and evaluates existing bridges using CHBDC. MI has also introduced their own design vehicle, the HSS-25, which is a configuration that must be considered alongside AASHTO live loadings for the design of Manitoba provincial highway bridges.

This technical note describes the evolution of highway bridge design live load models over the past five decades, focusing specifically on the case of Manitoba, Canada. The note presents this evolution within the context of major truck size and weight policy changes over the same period. In doing so, this note addresses the following two questions: *How have design trucks and lane loads evolved over the past five decades in Canada—specifically in Manitoba? How responsive have bridge design codes been to changes in truck size and weight policy?*

2.3 TRUCK SIZE AND WEIGHT LIMITS AND THE EVOLUTION OF BRIDGE LIVE LOAD MODELS: 1974 - CURRENT

This section presents a timeline of the evolution of highway bridge live load models within the context of truck size and weight regulatory changes, as depicted in Figure 1. Four bridge design codes are integral to this discussion: The *Ontario Highway Bridge Design Code* (OHBDC), the American Association of State Highway and Transportation Officials (AASHTO) *Standard Specifications for Highway* Bridges and *LRFD Bridge Design Specifications*, the Canadian Standard Association *Design of Highway Bridges* (CSA S6) and the *Canadian Highway Bridge Design Code* (CHBDC). Editions of these codes from 1970 to current will be introduced in this section.

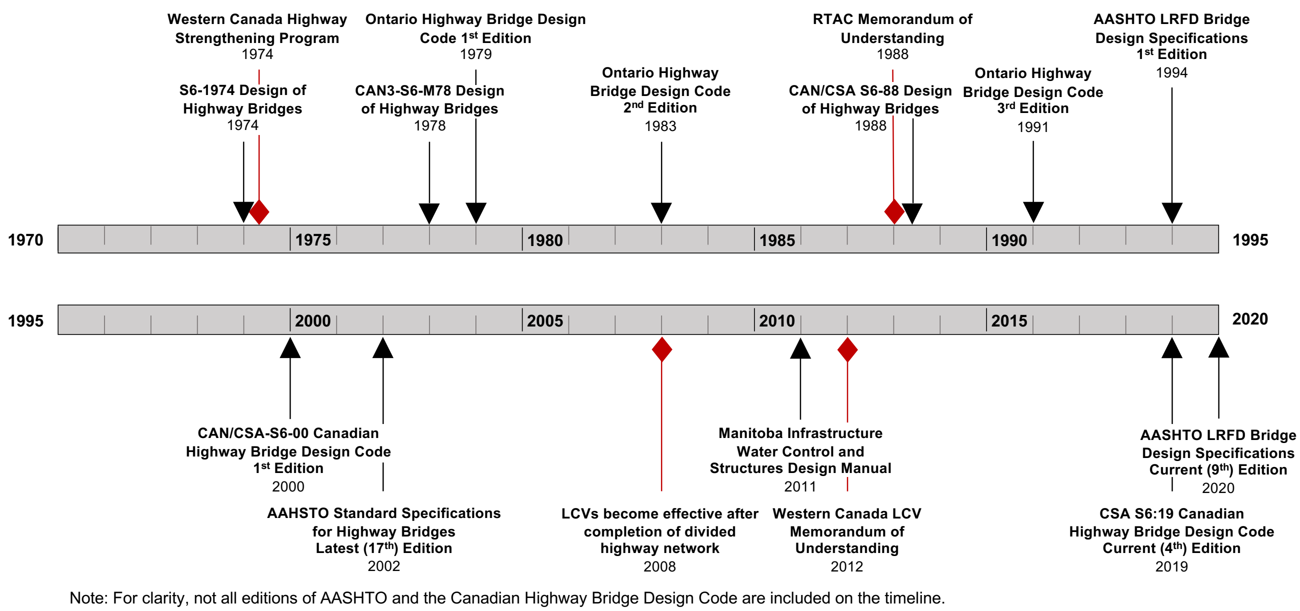

Figure 2-1: Timeline of critical bridge design code changes and regulatory changes 1970 – 2020

In general, this section serves as an introduction to the development of the live load models by presenting the design trucks and lane loads, and is not considered a comprehensive summary of the codes themselves. The development of the AASHTO design specifications and live load models over the past century has already been previously documented (Kulicki and Mertz 2006; Kulicki and Stuffle 2006); therefore, the following summary of the evolution of the highway bridge design live load models primarily focuses on the development of the Canadian codes. Moreover, this section does not address the evolution of the resistance side of the design model, nor does it address load and resistance factors, dynamic load allowance (or impact) effects, the number of design lanes, the impact of multiple loads both longitudinally and transversely (i.e., multiple presence factors), or the distribution of the load to an individual element (i.e., the girder distribution factor), all of which are important considerations for bridge design, code comparison and safety/reliability assessments.

2.3.1 1974: S6-1974 Design of Highway Bridges

Early editions of the CSA Standard *Design of Highway Bridges*, including the 1974 edition, specified the AASHTO *Standard Specifications* design vehicle systems: H loadings and HS loadings. The H loadings consist of a two-axle truck and are designated as HXX, where XX denotes the gross weight of the standard truck in US tons. H loadings were specified in the first edition of the AASHTO (previously AASHO) specification in 1931 and the H20 truck is representative of commercial vehicles around the year 1915 (Kulicki and Stuffle 2006).

The HS loadings consist of a three-axle tractor semi-trailer truck or the corresponding lane load and were introduced in the 1944 AASHTO *Standard Specifications.* This loading did not represent any particular truck, although its configuration was representative of a commonly used group of vehicles, five-axle semitrailers (3-S2s) (Kulicki and Mertz, 2006). Again, the HSXX designation is used, where XX indicates the gross weight in tons of the tractor. The limits of the axles appear to have been based on the 1956 U.S Congress limits, which allowed maximum axle weights of 18,000 lbs (80 kN) and 32,000 lbs (142 kN) for single and tandem axles respectively, unless larger loads were previously grandfathered in (Kulicki and Mertz 2006).

The variable trailer axle on the HS loading was introduced to better approximate truck trailers in use and allowed for flexibility in placement of the axle loadings to produce the maximum negative moment on a continuous span. Lane loads, expressed as a uniformly distributed load (UDL), were also included in the design code to account for multiple truck loading on the structure at a given time. The bridge was to be designed to the worst case loading between the truck and lane loading. Although the S6-1974 code was open to engineering judgement, highway bridge designs typically accommodated the H20 and HS20 trucks, as shown in Figure 2 (CSA 1974).

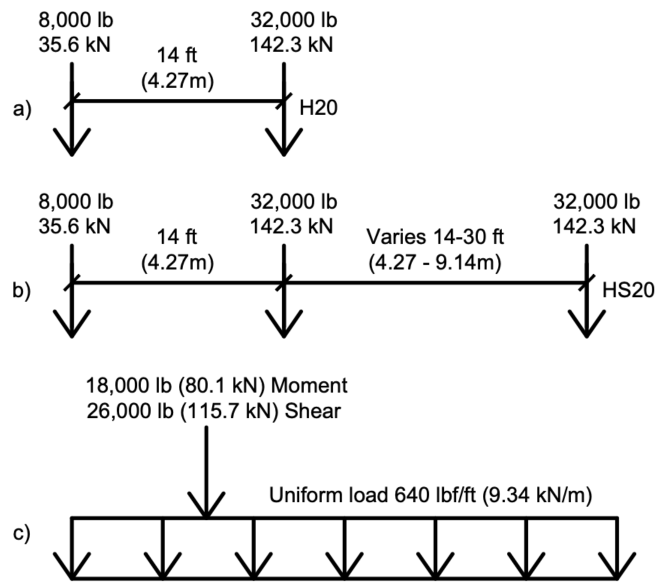

Figure 2-2: Live load models for: (a) standard H20 design truck, (b) standard HS20 design truck, and (c) H and HS lane loading (CSA 1974)

2.3.2 1974: Western Canada Highway Strengthening Program and Subsequent Derivatives

The 1974 WCHSP was the result of federal funding directed towards the Prairie Provinces (Alberta, Saskatchewan and Manitoba) to assist in the strengthening of pavements and bridges provided that these provinces allowed increased size and weight regulations on a primary road network. Following this program, the Prairie Provinces replaced the single/tandem/gross vehicle weights of 8,200/14,600/33,600 kg (80/143/330 kN) with 9,100/16,000/50,000 kg (89/157/490 kN) on primary highways. This change had an immediate effect on 3-S2 configurations and also permitted operation of six- or seven-axle double trailers (Regehr et al. 2009). The new allowable tandem weight limit now exceeded the unfactored HS20 design tandem axle weight.

In 1982, further weight increases occurred on both primary and secondary highways. These increases impacted seven-axle double trailers and eight-axle A and C-doubles, allowing them to operate at a GVW up to 56,500 kg (554 kN) in Manitoba (Regehr et al. 2009).

2.3.3 1978: CAN3-S6-M78 Design of Highway Bridges

In 1978, a revised *Design of Highway Bridges* code was released by CSA, which provided the design loads in metric and introduced a slightly modified design truck and lane loading. The release of the metric code was likely influenced by the Canadian construction industry's target year of 1978 for conversion to metric (Csagoly and Dorton 1978). The code specified that the highway live loading shall consist of an appropriate design vehicle, axle configuration and loading based on an analysis of actual vehicles and expected traffic situations (CSA 1978). But as an alternative to this, the metric conversion from US ton to tonne (MS loading), was introduced. The MS loading consists of a truck and lane load and could be used for the design of ordinary bridges. The axle spacings of this truck were similar to those of the previous HS20 model, but the axle weights and lane load UDL were increased for the MS200 and MS250, which were the loadings typically used, partially due to rounding after the metric conversion, as shown in Figure 3. It is unclear whether the increased axle weights of the new design truck were related to the allowable truck weight increases in 1974; however, the new unfactored bridge design tandem axle weights for the MS200 and MS250 were now comparable to or greater than the loads specified in the WCHSP.

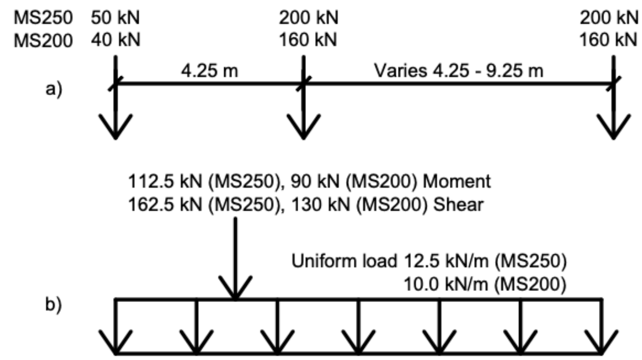

Figure 2-3: Live load models for: (a) standard MS200 and MS250 design trucks, and (b) lane loading (CSA 1978)

2.3.4 1979 - 1983: Ontario Highway Bridge Design Code 1st and 2nd Editions

Prior to 1979, the Province of Ontario used the standard HS live load model (Figure 2) provided in AASHTO for the design of bridges, despite truck weight limit increases in 1971 that permitted trucks to operate at a maximum weight of 623 kN following the Ontario Bridge Formula (Harman and Davenport 1979). Research in the 1970s focused on the development of a new LRFD code that used a live load model based on vehicle observations gathered from an extensive truck weight survey conducted in Ontario from 1967 to 1975 (Agarwal and Cheung 1987).

Using the survey results and a uniformly distributed load on an "equivalent base length" as a model for spans less than 25 m, the initial OHBD Truck configuration was developed, producing design loadings unlike the previous AASHTO model (Csagoly and Dorton 1978). This new live load model represented the maximum observed truck loads, including overloaded vehicles that exceeded the legal weight limits (Agarwal and Cheung 1987; Harman and Davenport 1979). The calibration was completed by selecting load and resistance factors and conducting a reliability analysis to achieve a target reliability index of β = 3.5, followed by design checks to compare bridges designed according to the previous AASHTO loadings (using WSD) to those designed using the new OHBDC and live load model (Csagoly and Dorton 1987). The OHBD Truck idealized axle weights and spacings did not represent any particular truck, although its configuration resembled a simplified version of double-trailer configurations (Csagoly and Dorton 1978).

During the initial development of OHBDC it was acknowledged that a complete set of statistical data was unavailable and modifications to the code and calibration would be required in future editions (Csagoly and Dorton 1978). The first edition of OHBDC was issued in 1979, was written in SI units, and was the state-of-the-art for LRFD bridge codes in North America.

After four years of experience in applying the code, it underwent revisions, clarifications and simplifications, leveraging additional research findings for the second edition in 1983. This work included dynamic testing programs to modify provisions in the code, studies to develop design provisions for wood bridges, and simplification of the methods of analysis and required number of load combinations (Dorton 1984). A limited vehicle survey conducted in 1979 indicated that the design vehicle, the OHBD Truck and lane load, still represented the truck population and consequently, the live load model did not change between the editions (Dorton and Bakht 1984), but the load and resistance factors were recalibrated during this time (Nowak and Agarwal 1981). The live load models are included in Figure 4 (OHBDC 1979; OHBDC 1983).

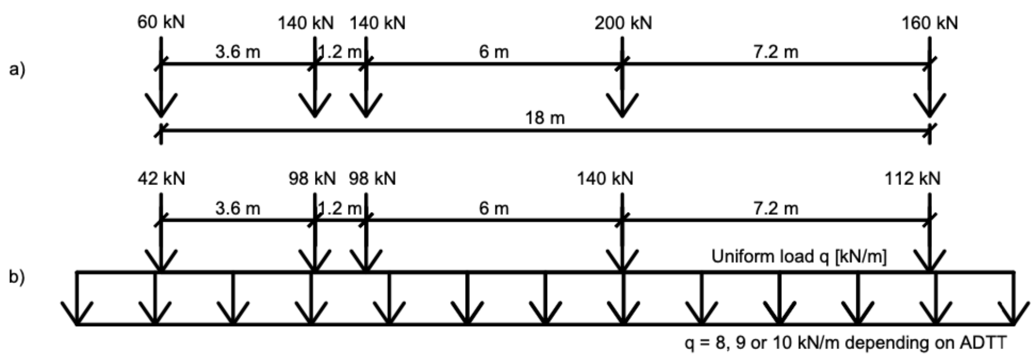

Figure 2-4: Live load models for: (a) OHBD Truck, and (b) lane loading (OHBDC 1979; OHBDC 1983)

2.3.5 1988: CAN/CSA S6-88 Design of Highway Bridges

In 1980, the CSA-S6 Code committee commenced a review of the 1978 live load model, taking into consideration the changes in legal weight limits and configuration of trucks that were currently operating on the Canadian interprovincial highway network. Following this review, it was identified that neither the CSA S6-78 MS200 loading model nor the AASHTO HS20 loading reflected the loads imposed by current-day trucks (Agarwal and Cheung 1987).

At this time, there was a proposal by the Council of Ministers to further increase the legal load limit across Canada and create uniformity in truck regulatory limits, understanding that previous studies had established that bridge capacity could allow for legal weight increases (Agarwal and Cheung 1987; Roads and Transportation Association of Canada 1980). In accordance with the expected increases, the unfactored CS-loading model was developed to be slightly heavier than the recommended weight limits and the model was then calibrated to meet the expected reliability index of β = 3.5 using truck weight and configuration data that was collected across the country. Although the OHBD live load model was found to perform well compared to the observed truck load effects, this model was not considered for the CSA code because it was based on maximum observed loads, including operational overloads, which were instead desired by the code committee to be accounted for with a live load factor (Agarwal and Cheung 1987). Additional details on the development of this live load model are outlined by Agarwal and Cheung (1987). The CS-600 live load model, as shown in Figure 5, was specified as the new standard design live load, resulting in a live load model significantly different than the previous CSA design codes (CSA 1988).

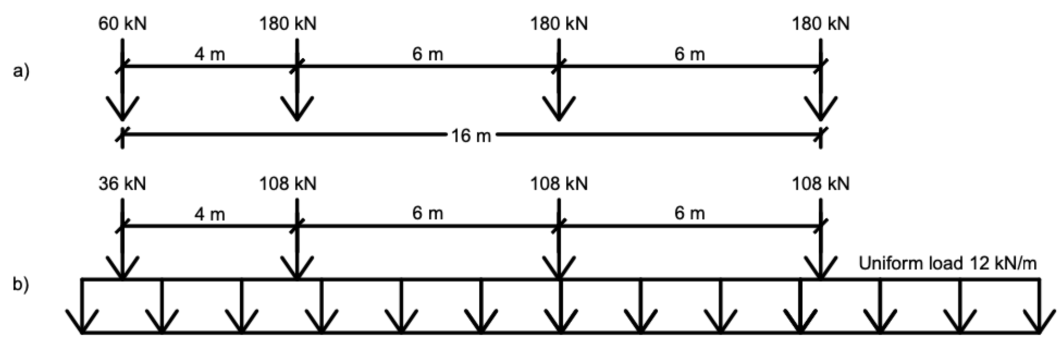

Figure 2-5: Live load models for: (a) CS-600 design truck, and (b) lane loading (CSA 1988)

2.3.6 1988: Roads and Transportation Association of Canada Memorandum of Understanding on Heavy Vehicle Weights and Dimensions and Subsequent Derivatives

As an extension of the previously proposed Council of Ministers' loading, a *Vehicle Weights and Dimensions Study* was conducted to develop preferred vehicle weights, dimensions, and configurations that could be used on major highways across Canada (Roads and Transportation Association of Canada 1986). The technical study analyzed and ultimately recommended implementation of a tridem axle group, which was previously not recognized in the Prairie Provinces (Woodrooffe et al. 2010). Based on the conclusions and recommendations of this study, a memorandum of understanding (i.e., the RTAC MoU) was signed nationally in 1988 for the purpose of creating national uniformity and facilitating interprovincial movement of freight in Canada.

A significant outcome of the MoU was the widespread implementation of the eight-axle B-double (3-S3-S2 configuration), which due to its superior dynamic performance characteristics was given a weight advantage over other double-trailer configurations making it suitable for hauling high density, weigh-out commodities (Regehr et al. 2009). Since the original signing, the RTAC MoU has undergone nine amendments with modifications to the allowable axle weight and dimension limits, one example being the increase of semitrailer lengths from 14.65 m (48 ft) to 16.2 m (53 ft) in 1994 (Council of Ministers of Transportation and Highway Safety 2019). Critical vehicle configurations that resulted from the RTAC MoU are summarized in Figure 6 for 5-axle tractor semitrailers (3-S2s), 6-axle tractor semitrailers (3-S3s), and 8-axle B-doubles (3-S3-S2).

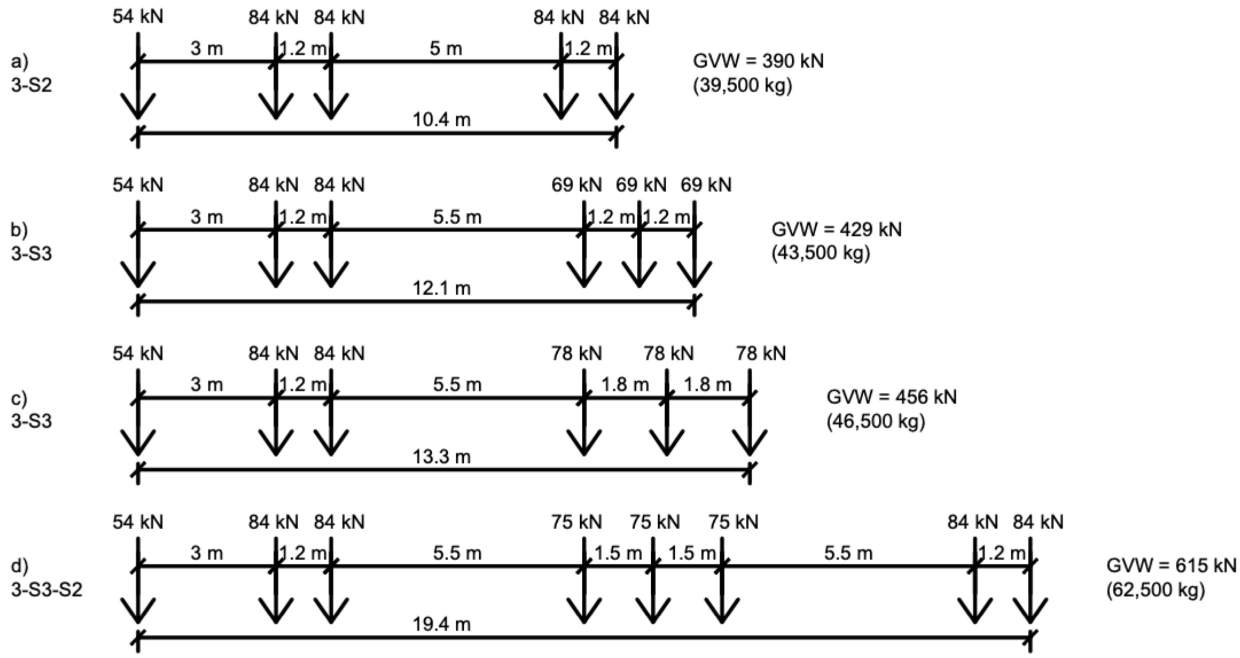

Figure 2-6: Critical vehicle configurations specified in the RTAC MoU

In Manitoba, as of June 2011, regulations under the Manitoba Highway Traffic Act were amended to allow a 1,000 kg (9.8 kN) increase to the allowable GVW on specified RTAC routes, including the entire Trans-Canada Highway in Manitoba. Prior to 2011, Manitoba was the only province in Canada that had not increased the B-double GVW limit from 62,500 kg (613 kN) (as originally specified in the RTAC MoU) to 63,500 kg (623 kN).

2.3.7 1991: Ontario Highway Bridge Design Code 3rd Edition

The RTAC MoU truck weight regulations developed a minimum standard for highways carrying inter-provincial traffic but understood that individual provinces may implement modifications to exceed these limits. This was the case in Ontario, which unlike Manitoba and many other provinces, had implemented its own truck size and weight regulations allowing trucks to operate up to a maximum GVW of 623 kN (63,500 kg) as early as the 1970s (Harman and Davenport 1979). As a result, Ontario truck operating loads were not meaningfully influenced by the regulation change and the third edition of OHBDC, and corresponding live load model, did not undergo modifications in direct response to the RTAC MoU.

In preparation for the third edition of OHBDC, a study was done to verify the load and resistance factors that were specified in the previous edition following further research in the material resistance models and dynamic loading. Additional structure types, including reinforced concrete T-beams, were also considered in this study. Similar to the previous editions, the live load model still referenced the 1975 truck survey dataset with the maximum 50-year load determined from extrapolation.

Following the reliability analysis of representative bridge types, including reinforced concrete T beams, designed to OHBDC 1983, shorter bridge spans less than 40 m were found to have a reduced safety reliability index (Nowak and Grouni 1994). To compensate for this, the design truck tandem axle was recommended to be increased from 140 kN to 160 kN in the 1991 edition, as shown in the Figure 7 live load models (Nowak and Grouni 1994; OHBDC 1991). Calibration was performed for the new load model and dynamic loading provisions and the material resistance factors were revised in order to meet the design reliability index of β = 3.5.

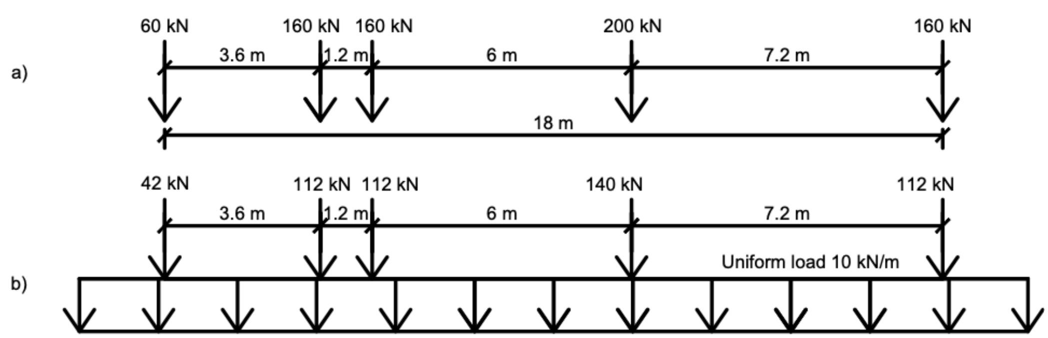

Figure 2-7: Live load models for: (a) OHBD Truck, and (b) lane loading (OHBDC 1991)

2.3.8 1994: AASHTO LRFD Bridge Design Specifications 1st Edition

As AASHTO transitioned from working stress design (WSD) and load factor design (LFD) to load and resistance factor design (LRFD) for the AASHTO *LRFD Bridge Design Specifications*, a new live load model was developed, beginning with consideration of regulatory and operational issues throughout the United States Interstate Highway System (Kulicki and Stuffle 2006). The regulatory environment was complex, allowing a variety of state legal loads, unanalyzed permit loads, and grandfather provisions with weight limits governed by the federal bridge formula (Kulicki and Mertz 2006). Grandfather rights allowed vehicles with heavier weights and modified axle spacings to operate beyond the legal limits, provided such operation was legal in the state prior to 1956 (Kulicki and Stuffle 2006). Over time, it became increasingly apparent that the HS20 live load model from 1944 did not represent the vehicles currently in operation.

A preliminary analysis was conducted to compare the moment and shear load effects of several existing truck configurations and new legal load proposals presented in the 1990 Transportation Research Board *Special Report 225* to the current HS20 design load for various span lengths (Kulicki and Mertz 2006; National Academies of Sciences, Engineering, and Medicine 1990). Of the proposed families of configurations, analysis showed that the exclusion vehicles that represented grandfathered loads currently used in several states produced the most severe load effects and were therefore selected as the basis for developing the new national design load (Kulicki and Mertz 2006).

Following the preliminary assessment, additional candidate live loads were proposed and compared to the family of exclusion vehicles and throughout this process, the HL-93 design loading, which includes the HS20 truck or a tandem load combined with a uniform distributed lane load (UDL), was developed, analyzed and calibrated based on an extrapolated version of the 1975 Ontario truck survey database. The tandem (military) loading was developed to account for more severe loading due to overweight single axles or axle groups on short spans. Figure 8 presents the HL-93 load models.

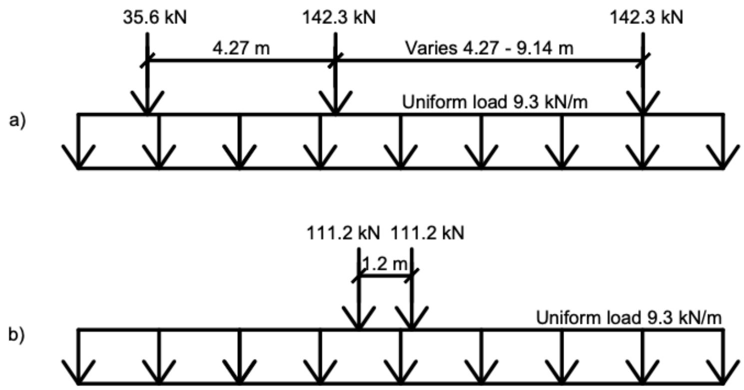

Figure 2-8: Live load models for: (a) HL-93 standard design truck and lane loading, and (b) design tandem and lane loading (AASHTO 1994)

Other papers have documented the evolution of AASHTO *Standard Specifications* and the development of the AASHTO *LRFD Bridge Design Specifications* and may be referenced for additional detail (Kulicki and Mertz 2006; Kulicki and Stuffle 2006). Despite the release of the LRFD code in 1994, the LFD code was still updated, and the latest edition of the *Standard Specifications* was issued in 2002 (AASHTO 2002).

2.3.9 2000: CAN/CSA-S6-00 Canadian Highway Bridge Design Code 1st Edition

In the 1990s, two Canadian-developed design codes were used for the design of highway bridges, OHBDC and CSA-S6. Both codes followed LRFD philosophy but had some differences in their approach to reliability analysis and calibration and were based on different truck size and weight regulatory environments. The development of the first edition of CHBDC started in 1992, expanding and adapting these two previous codes.

Considering the nationwide truck size and weight limits of the RTAC MoU, the new CL-W Truck live load configuration was created based on the B-double configuration, with the CL-625 model specified as the minimum standard for bridges across the nation. Figure 9 shows the CL-625 and the associated lane load. Although the unfactored GVW of the CL-625 is representative of the maximum legal limit used for B-doubles on a specified highway network (i.e., 63,500 kg), the axle distributions are different, and the model includes an overweight truck drive tandem axle heavier than the allowable limit and a heavy single axle instead of the tridem axle. These modifications were made to help reduce structural analysis computational effort, while providing a uniform reliability for all ranges of span lengths (especially short spans) and accounting for the fact that individual axles can be overloaded by as much as 100% of the allowable limit (CSA 2019).

To confirm consistency within the code and meet the accepted level of safety for the 75-year design life, the code was calibrated by performing a series of design checks and completing a reliability analysis using truck weight data collected in five Canadian provinces prior to the publication of the first edition of CHBDC in 2000.

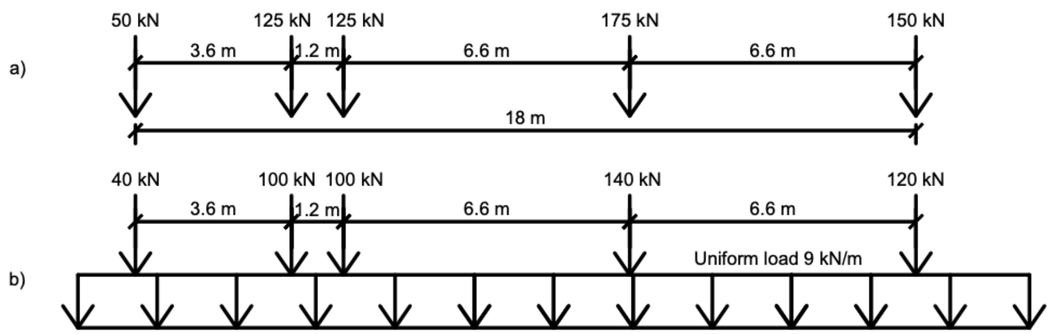

Figure 2-9: Live load models for: (a) CL-625 design truck, and (b) lane loading (CSA 2000)

2.3.10 2011: Manitoba Infrastructure Water Control & Structures Design Manual

Unlike most other Canadian provinces and the territories, Manitoba Infrastructure (MI) does not design its provincial highway bridges in accordance with CHBDC and instead follows the AASHTO LRFD code with an amended Structures Design Manual that was formally issued in 2011 (Manitoba Infrastructure and Transportation 2011). This manual provides guidance for the design and construction of bridges throughout the provincial network and specifies the use of the following three live load models for design: Modified AASHTO MSS 22.5 (HSS-25) Truck, AASHTO MS 27 (HS 30) Lane Load, and the AASHTO LRFD HL-93 model. As before, note that the MS loading nomenclature is simply the metric conversion from US ton to tonne. The HSS-25 truck is shown in Figure 10.

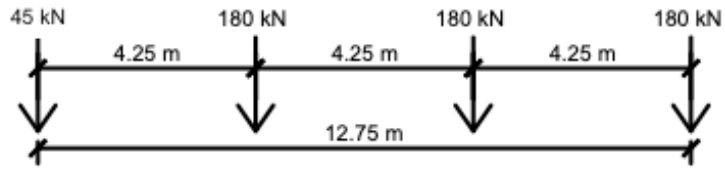

Figure 2-10: Live load model for Modified AASHTO HSS-25 Truck (Manitoba Infrastructure and Transportation 2011)

Based on correspondence with MI department engineers, the Modified HSS-25 loading model was originally developed in the mid-1980s by adapting the AASHTO HS20 model, increasing it by 1.25 times (i.e., 25/20) and adding an additional trailer with the same inter-axle spacing (A Pankratz, personal communication, 2020). This model is not used in conjunction with a lane load. Use of this model for design has been found by MI to be beneficial when evaluating existing structures for permit and overload vehicles (A Pankratz, personal communication, 2020). Despite this practical value, there may be a need to formally evaluate the representativeness of this live load model to Manitoba traffic, to verify the calibration of the new load model with the provided load and resistance factors specified in AASHTO, and to compare the HSS-25 model with previously established CHBDC or AASHTO models with design checks. A study previously conducted in Manitoba used a rational method and observed truck traffic to determine a Manitoba-specific CL-W (CL-555) design live load in accordance with CHBDC (Algohi et al. 2019).

2.3.11 Ongoing Permitting of Longer Combination Vehicles

LCVs are truck configurations consisting of a tractor and two or three trailers or semi-trailers that exceed basic vehicle length limits but remain in compliance with the GVW limits identified in the RTAC MoU for vehicles with eight or more axles (Wood and Regehr 2017). Operations of LCVs occur on a specified portion of the divided highway network. In 2008, the completion of the divided highway network in the Prairie Provinces allowed for uninterrupted operation of LCVs between major urban centers, resulting in more effective use of these vehicles (Wood and Regehr 2017). In 2012, an agreement between the governments of British Columbia, Alberta, Saskatchewan, and Manitoba developed standards for the operation of LCVs in those provinces, demonstrating the desire to create an even more cohesive interprovincial transportation environment. This MoU allowed for A-, B-, and C-coupled turnpike double (3-S2-4) configurations to operate at a 63,500 kg (623 kN) GVW limit on a specified road network (Wood and Regehr 2017). Coincident with these policy changes, LCV use increased and represented 10% of the articulated truck fleet in Manitoba in 2013 (Wood and Regehr 2017).

Although LCVs are not operating at heavier weights than B-Doubles, the configurations are longer, which potentially introduces different loading conditions to bridges for negative moment ultimate limit states and fatigue serviceability limit states than the typical RTAC truck fleet does. Considering the unique loading conditions and the fact that the level of use of LCVs has increased substantively over the past 15 years (Wood and Regehr 2017), bridge live load models in Canada may require assessment in response to modifications in LCV truck size and weight policies over the past decade.

2.3.12 Current Bridge Design Codes

The current bridge design codes in North America are the 2020 AASHTO LRFD 9th Edition and 2019 CHBDC 4th Edition (AASHTO 2020; CSA 2019). The live load models have not been changed from the models that were calibrated for the initial releases in 1994 and 2000 (i.e., the HL-93 and the CL-625). In 2007, a study was completed following a request by AASHTO to document and review the assumptions required for the first AASHTO LRFD edition calibration, confirm adequacy of the calibration, and conduct a sensitivity analysis for the load and resistance factors for an expanded suite of bridges nearly 15 years later (Kulicki et al. 2007). Following this study, the parameters used for code calibration were documented and although Monte Carlo simulation was found to be the most robust method for future calibration work, the sensitivity analysis returned values in close agreement to the previous calibration results for the revised bridge database. No changes to the AASHTO LRFD design code resulted from this project.

2.4 CONCLUDING REMARKS

Truck size and weight regulation modifications in Canada have influenced the interprovincial freight operating environment over the last five decades, which in turn has impacted the truck fleet mix, trucking costs and rates, shipment sizes and truck gross vehicle weights (Clayton and Lai 1986; Clayton and Nix 1986; Regehr et al. 2009). As presented in this article, these policy changes have also influenced modifications to bridge design codes and live load specifications. The evolution of load and resistance factor design live load models in North America was presented to document how the design truck and lane loads have changed since the 1970s and explain the path that led to the development of the first edition of CHBDC and the CL-625 load model. Manitoba-specific commentary was presented because of the province's interesting historical regulatory environment, its unique decision to consider both AASHTO and CHBDC design codes, and its specification of a different design vehicle, the HSS-25.

The relationship between truck size and weight regulations and bridge design codes has historically been complex. Key observations, which collectively address the two research questions raised at the outset of this note, follow:

- Despite truck size and weight increases in 1974 in the Prairie Provinces, it is unclear whether the subsequent 1978 CSA-S6 design code increased the live load model in response to these changes. However, it was documented in studies in the early 1980s that the MS-200 model did not reflect current-day heavy trucks in Canada, prompting research into a new live load model.
- The Province of Ontario also increased legal truck weight limits in the early 1970s and the OHBDC was published afterwards in 1979, following a detailed truck weight survey and study of observed vehicle load effects in 1975.
- The weight increases articulated in the 1988 RTAC MoU appeared to have affected provinces and design codes differently depending on the previously allowed weights. Ontario trucks were already operating near this limit and no changes directly related to the RTAC MoU were warranted in the 1991 OHBDC.

In contrast, Manitoba truck weight limits were increased significantly following the RTAC MoU; however, the 1988 CSA S6 code and similar studies pre-emptively evaluated the effects of the proposed Council of Minister loadings and adjusted the bridge design live load model. The next bridge code was not issued in Canada until CHBDC in 2000 and it presented a new live load model, the CL-625, that was directly related to the RTAC MoU legal loads.

- Although the United States truck size and weight regulations have undergone relatively little change over the past five decades, influences of heavy grandfathered vehicles operating on the federal highway network prompted an assessment and revision of the live load model for the release of the AASHTO LRFD code in 1994.
- Following several policy changes involving LCV operations and the expansion of an effective highway network to enable their widespread use, these unique truck configurations may require future assessments to determine if the current design live load models appropriately envelope the observed load effects.
- The uncalibrated design load in Manitoba, the HSS-25, uniquely evolved as a way to represent truck loads in the province while enabling the use of the AASHTO LRFD code. Despite general satisfaction with this approach, there appears to be a need to formally assess the level of safety of this live load model.

The historical review of truck size and weight policy and bridge design code modifications reveals instances in which bridge codes clearly and responsively accounted for truck weight policy changes, as well as situations when code modifications lagged regulatory change by several years. While eventual evaluations of these situations have not necessitated major code revisions, the review nevertheless highlights the importance of a collaborative relationship between policy makers and bridge code authors to ensure a consistent level of safety for trucks operating on bridges in North America.

2.5 ACKNOWLEDGEMENTS

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3.0 **Retrospective Longitudinal Study of the Impact of Truck Weight Regulatory Changes on Operating Gross Vehicle Weights**

To assess the impacts of TSW regulations on bridge infrastructure and other key areas such as safety, productivity and emissions, an understanding of the expected changes to truck operating weights is required. Although *ex ante* analysis methods are available to model and predict the changes in truck weights for various truck configurations, difficulties in validating these estimates have historically limited this approach. Alternatively, *ex post* research, as is presented by the forty-five year longitudinal study in the following paper, allows for unique insights to be gained from the observed truck operator responses. In particular, the results show how and when the truck fleet and GVW distributions changed in concert with allowable weight limit modifications in Manitoba, Canada. As such, this chapter addresses the second research question: How have truck gross vehicle weights evolved as a function of truck weight limits?

The material in this chapter was submitted for publication in the Transportation Research Record on October 9, 2020 and is printed here with permission of co-author Jonathan Regehr. The paper is self-contained in the chapter with its own abstract, introduction, conclusion and references. The author of this thesis was the lead author, researcher and analyst, and had principal responsibility for all aspects of the paper, while the co-author provided advice and reviews.

3.1 ABSTRACT

Three primary truck size and weight policy changes occurred in Canada over the past five decades: the 1974 Western Canadian Highway Strengthening Program, the 1988 Roads and Transportation Association of Canada Memorandum of Understanding on Heavy Vehicle Weights and Dimensions, and ongoing special permitting of longer combination vehicles (LCVs). These regulatory changes influenced the gross vehicle weights (GVWs) of predominant truck configurations operating on principal Canadian highways. Using a unique time-series of truck weight data, this retrospective longitudinal study contributes insights about the magnitude and timing of the impacts of truck weight regulatory changes on operating GVWs that address current knowledge gaps and

persistent uncertainties in models used to predict and evaluate truck weight regulatory changes.

The analysis reveals that carriers hauling heavy (i.e., weigh-out) commodities adapt immediately to increases in GVW limits if there is no need to purchase new vehicles. When a regulatory change coincides with the introduction of a new, more productive vehicle configuration, the uptake of the new vehicle lags the regulatory change by a few years. Finally, configurations exhibit different GVW distributions and responses to increased GVW limits depending on whether the configurations are well suited for hauling weigh-out or cube-out commodities. This differential response demonstrates how regulations facilitate fleet diversity within the trucking industry’s approach to the road freight transport task.

3.2 INTRODUCTION

Over the past several decades, the trucking industry has responded to considerable growth and change in the demand for road freight transport, the availability of new technologies, and a complex and ever-evolving regulatory environment. Truck regulations define and control, *inter alia*, truck size and weight (mass), dynamic vehicle performance, safety requirements, vehicle and driver credentials, energy use, and emissions. Despite the ongoing need to modify existing or introduce new truck regulations, research and practice consistently demonstrate the difficulty of predicting and understanding the impacts of proposed regulatory changes regardless of the regulatory context [1] [2] [3].

Governments contemplate changes—usually increases—in truck size and weight limits to increase truck productivity and minimize regulatory barriers between jurisdictions. These outcomes must be considered alongside the positive or negative impacts on vehicle and system safety, infrastructure (bridges, pavements, and road geometry), energy use and emissions, modal competition and diversion, and regulatory compliance [4] [5] [6] [7] [8].

An *ex ante* analysis of the various impacts of truck size and weight regulatory change relies on the ability to model future truck activity in terms of the total distance travelled by prospective truck configurations, the networks on which such travel will occur, and the distribution of operating weights of these configurations [6] [7]. Previous studies have developed truck activity estimates through a detailed characterization of freight transport demand, a modal diversion analysis, and the estimation of the number and nature of truck trips required to meet future demands. The reliability of these methods for estimating the effect of policy changes has been questioned, calling for research that addresses methodological limitations and validates assumptions and predictions using empirical data [9] [1]. Naturally, uncertainties in truck activity estimates propagate through to the analysis of the impacts of regulatory changes. In some cases, these uncertainties have precluded policy action [3]. Consequently, the development of improved technical approaches to predict, validate, and understand the significance and outcomes of truck size and weight policy changes is both relevant and essential.

Given the limitations and uncertainties of *ex ante* analyses, a recent Transportation Research Board (TRB) *Consensus Study Report* calls for *ex post* research that documents empirical evidence of truck activity following truck weight regulatory changes [1]. Citing an earlier TRB study published in 2002, the *Consensus Study Report* asserts that “responsible regulation is a process: the regulatory authority should do the best prior analysis possible, but once regulations have been changed, the consequences must be systematically observed and adjustments made where necessary” [3]. While such systematic observation may be limited by the inability to control exogenous variables, given a suitable time-series of data, it could produce insights to help refine predictive modelling approaches.

This article presents an *ex post* analysis of the impact of historic truck weight regulatory changes on the operating gross vehicle weights (GVWs) of predominant truck configurations in Canada. In doing so, the article addresses the following research questions: *How do operating GVW distributions change when axle and GVW limits are relaxed? How do these changes differ for predominant truck configurations? How long does it take for the trucking industry to respond to truck weight regulatory changes?* In part, the article answers the call issued by the TRB in 2002 and reiterated in its most recent elucidation of truck size and weight research needs [1] [3].

The retrospective analysis utilizes a longitudinal truck weight dataset collected in Manitoba, Canada over the past 45 years. In this time, Manitoba and its neighbouring western Canadian provinces have experienced several major truck weight policy changes. Thus, the analysis reveals observed changes in truck operating weights without attempting to model these changes. Although the results stem from data collected in Manitoba, the general trends and observations provide insight to policy makers in other jurisdictions.

The remainder of this article is structured as follows. Section 3 synthesizes the literature on the impacts of truck weight regulatory change on truck operating weights. Section 4 outlines the historic truck weight regulatory changes in Manitoba, Canada. Section 5 describes the data sources and data screening procedures. Section 6 presents and discusses the results of the analysis of truck operating weights for predominant truck configurations. Section 7 outlines limitations of the analysis. The last section summarizes key contributions and presents concluding remarks.

3.3 LITERATURE REVIEW

In 2019, the TRB published a *Consensus Study Report* [1] that comprised a research plan to address limitations in the data and impact models that had affected the reliability of previous national-scale truck size and weight study results [6] [7]. The plan is the latest in a series of TRB *Special Reports* that have evaluated proposals and issues related to truck size and weight regulations in the United States [10].

- In 1986, TRB's *Special Report 211* evaluated industry utilization of twin trailer trucks and their safety and infrastructure consequences. These vehicles became widespread in the United States in the early 1980s [11].
- As a follow-on from the 1986 study, *Special Report 223* examined the issue of access from terminals to the primary network for large trucks with regard to vehicle handling (safety) and pavement impacts [12].
- In 1990, *Special Report 225* examined the implications of eliminating existing grandfather provisions on highways, removing GVW and axle limits in favour of a bridge formula, adjusting the existing bridge formula, and modifying treatment of

specialized hauling vehicles for seven alternative scenarios. The study recommended a new bridge formula, the implementation of special permit programs, the retention of grandfather rights, increased enforcement, and more standardized limits at a regional level [13].

- Also in 1990, *Special Report 227* presented the Turner Proposal which envisioned new truck configurations that would reduce pavement wear and increase transportation efficiency by increasing the number of axles and the GVW [14].
- In 2002, *Special Report 267* recognized that opportunities existed to improve the efficiency of the highway freight transport system by reforming federal truck size and weight regulations, which had remained unchanged for more than a decade. It noted that previous studies (e.g., [13] [6]) had not produced satisfactory estimates of the effects of regulatory change, but determined that the inevitable uncertainty should not justify inaction. The study recommended organizational arrangements to promote change in the current regulations, including pilot studies, federal regulatory changes, and additional research [3].

A persistent shortcoming identified in these studies is the need to predict changes in truck configurations and weights, axle loadings, traffic operations, and the total number of trips and distance required to haul freight types resulting from regulatory changes [7]. Comprehensive truck size and weight analyses rely on mode shift models (e.g., based on economic elasticities) to predict the future modal share of the freight transport task allocated to trucks. Such analyses subsequently distribute freight between prospective truck configurations based on available evidence (e.g., pilot studies) and stated preference surveys. However, the predicted allocation of freight to various truck configurations and the resulting operating GVWs are difficult to validate. This leads to uncertainties in the estimates of future activity and concomitant impacts on infrastructure [15] [7], operations and safety [16] [7], energy and the environment [17] [18], and regulatory compliance [7]. Thus, despite considerable previous research, the 2019 *Consensus Study Report* reiterates the need for data and models that could more accurately estimate the impacts of changing truck size and weight limits and support more meaningful policy action.

While the TRB's research plan focuses specifically on needs in the United States, it recognizes the potential value of drawing from international experience. Useful insights may be learned from thorough study of research and practice related to truck size and weight regulations in European nations, Australia, and elsewhere [4] [19] [20]. However, a retrospective analysis of the Canadian experience—as will be provided in this article—is perhaps most instructive as a response to the call in the 2019 *Consensus Study Report*, given Canada's geographic proximity to the United States [8].

In Canada, several truck size and weight regulation changes over the past five decades have impacted the GVW distributions of trucks operating on primary highways across the country and especially in the Prairie Provinces. According to Woodrooffe et al. [8], the implementation of these policy changes has not only increased truck operating weights, but has also harmonized the regulatory environment, improved trucking productivity, and encouraged the utilization of configurations that meet dynamic performance requirements.

Previous studies on the Canadian truck regulatory changes have presented observations on changes in truck fleet, truck costs and rates, shipping sizes and initial trucking industry responses [21] [22] [23]. Other studies examined the physical and loading characteristics of trucks between 1974 and 1984 [24] and developed models of the GVW and axle weight distributions as a function of weight limit over the 1972 to 1986 timeframe [25]. However, none of this work examined impacts of more recent changes in GVW distributions, as will be presented in this article

3.4 DESCRIPTION OF REGULATORY CHANGES IN MANITOBA, CANADA

In Canada, three primary truck size and weight regulatory changes have influenced trucking activity over the past five decades: (1) the 1974 Western Canada Highway Strengthening Program (WCHSP) (and its derivatives), (2) the 1988 Roads and Transportation Association of Canada Memorandum of Understanding on Heavy Vehicle Weights and Dimensions (RTAC MoU), and (3) regional policies regarding routine special permitting of LCVs [22]. In addition to these primary policy changes, the Province of Manitoba, like its provincial counterparts, has implemented changes to the allowable GVWs on specified portions of its highway network.

For the purpose and scope of this longitudinal study, this section provides a brief discussion of the allowable GVWs resulting from these regulations. Woodrooffe et al. provide a detailed description of the Canadian experience with truck regulatory changes [8]. Table 1 summarizes the maximum allowable GVWs over time and shows schematics of the four truck configurations studied in this article: (1) a five-axle tractor semitrailer (3-S2), (2) a six-axle tractor semitrailer (3-S3), (3) an eight-axle B-double (3-S3-S2), and (4) a turnpike double (typically comprising nine axles in a 3-S2-4 configuration, as shown). These four configurations were selected for the analysis because they represent a significant portion of the current truck fleet in Manitoba and have each been impacted by truck weight regulation changes.

Table 3-1: Historical Gross Vehicle Weight Limits (in kg) by Truck Configuration in Manitoba

Year \ Configuration	**3-S2** 	**3-S3** 	**3-S3-S2** 	**3-S2-4** 
Pre-1974	33,600	- [a]	- [a]	- [a]
1974 [b]	**37,500**	- [a]	- [a]	- [a]
1988 [c]	**39,500**	**46,500**	**62,500**	- [a]
1995 [d]	39,500	46,500	62,500	**62,500**
2011	39,500	46,500	**63,500**	62,500
2012	39,500	46,500	63,500	**63,500**
2014	**40,000**	**47,000**	63,500	63,500
Current (2020)	40,000	47,000	63,500	63,500

Notes:
Bold cells denote changes in gross vehicle weight limit for the year shown.
[a] The hyphen (-) indicates that the configuration was not commonly operated or not permitted to operate in that year.
[b] The Western Canadian Highway Strengthening Program (WCHSP) occurred in 1974.
[c] The Roads and Transportation Association of Canada Memorandum of Understanding on Heavy Vehicle Weights and Dimensions (RTAC MoU) was signed in 1988.
[d] Turnpike doubles (3-S2-4s) were first permitted to operate in Manitoba. They operate by special permit on a specified highway network [26].

3.4.1 1974 Western Canada Highway Strengthening Program and Subsequent Derivatives

The 1974 WCHSP was the result of federal funding directed towards the Prairie Provinces (Alberta, Saskatchewan, and Manitoba) to assist in the strengthening of pavements and bridges provided that these provinces allowed increased size and weight regulations on a primary road network. Following this program, the Prairie Provinces replaced the single/tandem/GVWs of 8,200/14,600/33,600 kg with 9,100/16,000/50,000 kg on primary highways. This change had an immediate effect on 3-S2 configurations and also permitted six- or seven-axle double trailers. In 1982, further weight increases occurred on both primary and secondary highways. These increases impacted seven-axle double trailers and eight-axle A and C-doubles, allowing them to operate at the full GVW limit obtained by summing the allowable axle weights. While somewhat prominent at the time, relatively few of these configurations remain operational today and therefore these configurations were not considered in this article or in Table 1 [22].

3.4.2 1988 Roads and Transportation Association of Canada Memorandum of Understanding on Heavy Vehicle Weights and Dimensions and Subsequent Derivatives

In an attempt to create national uniformity and facilitate interprovincial movement of freight in Canada, a Vehicle Weights and Dimensions Study was conducted to develop preferred vehicle weights, dimensions, and configurations that could be used on major highways across the country [27]. Based on the conclusions and recommendations of this study, a memorandum of understanding (i.e., the RTAC MoU) was signed nationally in 1988.

To further increase axle weights beyond those allowed by the WCHSP, the technical study analyzed and ultimately recommended implementation of a tridem axle group, which was previously not recognized in the Prairie Provinces [8]. The tridem axle was limited to a maximum weight of 21,000 kg for an axle spread of 2.4 m to 3.0 m, 23,000 kg for an axle spread of 3.0 m to 3.6 m, and 24,000 kg for an axle spread of

3.6 m to 3.7 m [28]. It could be added to a semi-trailer (to form a 3-S3 configuration) or used as the coupling device for a double-trailer configuration (the eight-axle B-double, which is normally configured as 3-S3-S2). The study found this eight-axle B-double to have superior dynamic performance and therefore these configurations were given a weight advantage over A- and C-double configurations in the RTAC MoU [8]. As a result, B-doubles became one of the more prominent configurations in the truck fleet after the RTAC MoU [22].

As shown in Table 1, two amendments to the RTAC MoU have increased allowable GVW limits in Manitoba since 1988. First, in June of 2011, regulations under the Manitoba Highway Traffic Act were amended to allow a 1,000 kg increase to the allowable GVW on specified RTAC routes in Manitoba, including the entire Trans-Canada Highway. Prior to 2011, Manitoba was the only province in Canada that had not allowed the B-double GVW limit increase from the 62,500 kg specified in the RTAC MoU to 63,500 kg. Since 2011, an increasing number of routes have been added to the "Super-RTAC" network [29]. Second, a June 2014 amendment allowed for a 500 kg increase to the maximum allowable steering axle weight. This increase was intended to offset additional weight for exhaust emission control technologies, wildlife protection, and other equipment added to the truck itself and therefore may not induce a meaningful payload response.

3.4.3 Ongoing Permitting of Longer Combination Vehicles

LCVs are truck configurations consisting of a tractor and two or three trailers or semi-trailers that exceed basic vehicle length limits but remain in compliance with basic weight restrictions [26]. This article focuses on turnpike double configurations only, which started operating in Manitoba after 1995 [30]. These operations occur on a specified portion of the divided highway network subject to the GVW limits identified in the RTAC MoU for vehicles with eight or more axles. Since 1995, turnpike doubles have represented an increasing proportion of the overall LCV fleet operating on the Trans-Canada Highway west of Winnipeg [31] and are the most widely permitted LCV configuration [26]. In 2012, an agreement between the governments of British Columbia, Alberta, Saskatchewan, and Manitoba developed standards for the operation of LCVs in those provinces, demonstrating the desire to create a more cohesive interprovincial transportation environment. This MoU allowed for A-, B-, and C-coupled turnpike double (3-S2-4) configurations to operate at a 63,500-kg GVW limit on a specified road network that connected most of the major urban centres in western Canada [26].

3.5 DATA

3.5.1 Data Sources

The data presented in this longitudinal study includes both published and unpublished on-road truck surveys conducted since the early 1970s by the Manitoba provincial government and the University of Manitoba Transport Information Group (UMTIG) at the Headingley and West Hawk weigh scales. These surveys were conducted for various research purposes and had differing durations and data collection times. As shown in Figure 1, the Headingley weigh scale is located approximately 7.5 km west of Winnipeg on the Trans-Canada Highway (PTH 1). This scale processes interprovincial and intra-provincial truck traffic and is a major link for turnpike double truck operations in Western Canada. The West Hawk weigh scale is located approximately 1.2 km west of the Manitoba-Ontario border on PTH 1. Most of the traffic moving through this scale is interprovincial and trucks must comply with both the Manitoba and Ontario truck size and weight regulations. Because Ontario prohibits the use of LCVs in this region, no turnpike doubles were observed. A permanent, static weigh scale is also located in Emerson; however, this article focuses on the truck traffic movements on PTH 1.

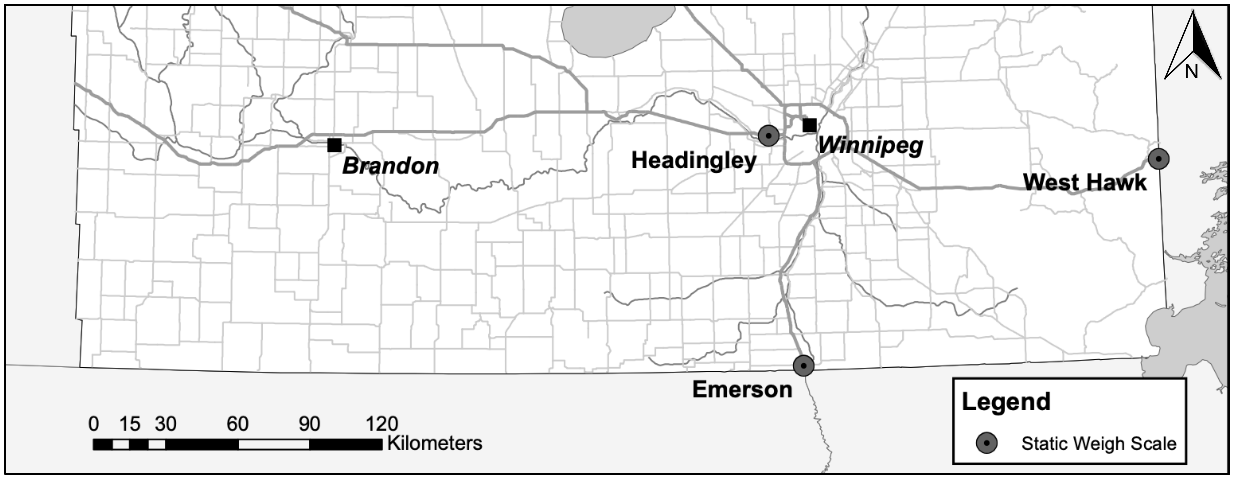


Figure 3-1: Static Weigh Scale Locations in Manitoba, Canada

For the analysis presented in this article, any truck survey dataset with more than 100 records for the sample year was included in the analysis, regardless of whether the data was collected at the Headingley or West Hawk scale. In total, the analysis included 32,147 GVW records collected over 17 sample years at the two scales. The dataset comprised 23,530 3-S2s, 5206 3-S3s, 2708 eight-axle B-doubles, and 703 Turnpike doubles. When available, the data collected in a given year was combined from these two weigh scales to provide a larger sample size.

3.5.2 Data Screening and Analysis Methods

Throughout the study period, a truck's GVW was consistently determined by recording and summing the weights of individual axle groups. These axle group weight readings require user input and are therefore subject to normal human error. The datasets were screened to prevent unrealistic values from skewing the GVW distributions. Following the same criteria used in previous studies [32], axle group weights less than 1,500 kg and more than 150% of the allowable load limit were deleted from the dataset. Only entries with valid axle group weights were considered in the analysis.

There is also potential for mistakes in classifying vehicles. For example, from 2000 to 2002, there were a small number of 3-S2s with unusually high rear tandem axle loads, suggesting that these may have actually been 3-S3s. Unless an error could be confirmed, records suspected of containing such errors remained in the dataset for subsequent analysis.

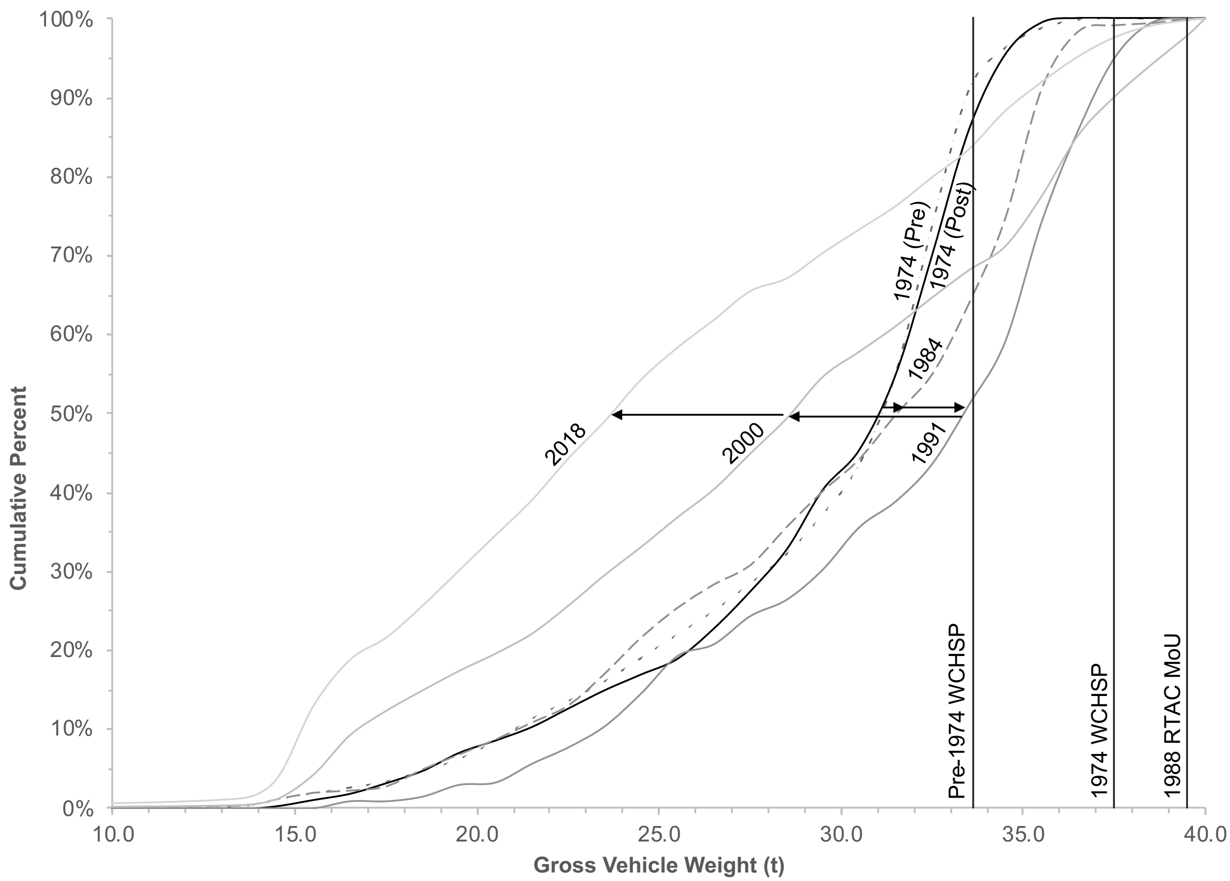
To test for differences between the GVW distributions, under a null hypothesis that both samples came from a population with the same distribution, the two-sample Kolmogorov-Smirnov test was used at a significance level of α = 0.05.

3.6 OBSERVATIONS AND INSIGHTS

This section presents observations and insights about the impact of truck weight regulatory changes on the GVW distributions for the four truck configurations identified in Table 1, namely: (1) five- and six-axle tractor semitrailers, and (2) eight-axle B-doubles and turnpike doubles.

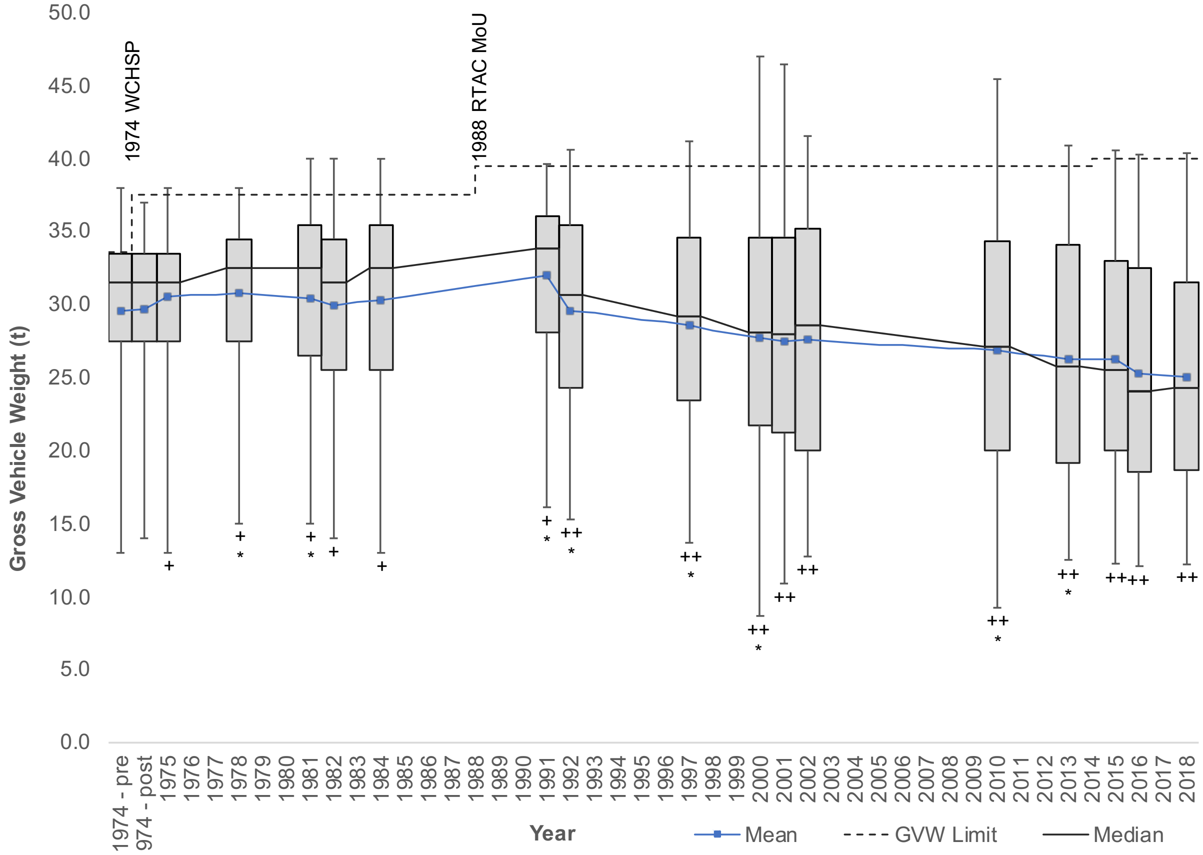
3.6.1 GVW Distributions for Five- and Six-Axle Tractor Semitrailers (3-S2s and 3-S3s)

In the early 1970s, 3-S2s (and other single-trailer configurations with five or fewer axles) comprised nearly the entire articulated truck fleet in Manitoba. While this configuration remains the most common type of articulated truck in Manitoba, it currently comprises approximately half of the trucks operating on the Trans-Canada Highway [22]. The GVW limit for 3-S2s in Manitoba increased three times during the study period: in 1974, 1988, and 2014. Figure 2 shows the GVW cumulative frequency distributions from selected years ranging from 1974 to 2018 and Figure 3 presents the box-and-whisker plots for the available GVW data.



Notes: 1) The 1974 distributions include all laden combinations operating on a primary Manitoba highway. 2) The 1984 distribution relates to a survey conducted at the West Hawk scale, but excludes observations made during winter (Dec. 1 – Feb. 28). 3) The distributions from 1974 – 1984 include laden vehicles only; surveyed trucks with no payload were excluded. 4) The 1991 distribution relates to a survey conducted at the Headingley scale. 5) The 2000 distribution relates to a survey conducted at both the West Hawk and Headingley scales. 6) The 2018 distribution relates to a survey conducted at the West Hawk scale.

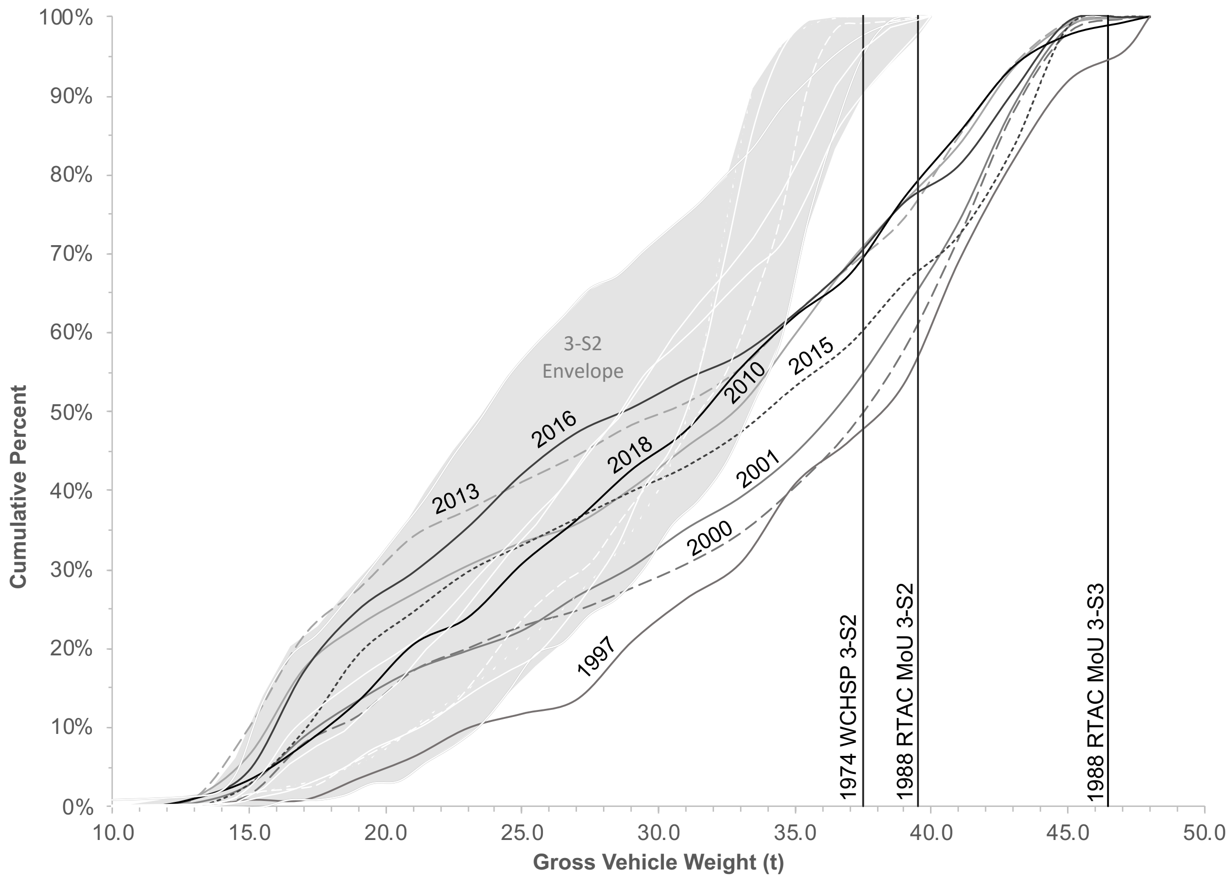
Figure 3-2: Cumulative Frequency Distributions for 3-S2 GVWs in Manitoba: 1974 - 2018



Notes: 1) The whisker lines represent the maximum and minimum gross vehicle weight values and the shaded box represents the interquartile range. 2) * indicates a significant two-sample Kolmogorov-Smirnov test at $\alpha = 0.05$ relative to the previous year's distribution. 3) + indicates a significant two-sample Kolmogorov-Smirnov test at $\alpha = 0.05$ relative to the pre-1974 distribution. 4) ++ indicates a significant two-sample Kolmogorov-Smirnov test at $\alpha = 0.05$ relative to the 1991 distribution.

Figure 3-3: Box-and-Whisker Plots for 3-S2 GVWs in Manitoba: 1974 – 2018

Unlike 3-S2s, 3-S3s only emerged in the truck fleet following the introduction of tridem axles pursuant to the RTAC MoU in 1988. Although data availability has limited the 3-S3 study period to the years 1997 to 2018, previous research showed the emergence of 3-S3s in 1989 and by 1997, 3-S3s represented approximately 25% of Manitoba's articulated truck fleet operating on the Trans-Canada Highway [22]. The GVW limit for 3-S3s increased once since their introduction: a 500-kg increase in 2014. Figure 4 shows the 3-S3 GVW cumulative frequency distributions during the study period. To provide context, the figure includes the 3-S2 GVW distribution envelope and the relevant 3-S2 and 3-S3 GVW limits. The 3-S3 GVW limit shown in the figure indicates the absolute maximum GVW based on a tridem axle limit of 24,000 kg.



Notes: 1) The 1997 and 2018 distributions relate to a survey conducted at the West Hawk scale. 2) The 2015-2016 distributions relate to a survey conducted at the Headingley scale. 3) The remaining distributions relate to a survey conducted at both the West Hawk and Headingley scales. 4) The shaded 3-S2 envelope represents the maximum and minimum cumulative percent for a given gross vehicle weight for the years 1974-2018.

Figure 3-4: Cumulative Frequency Distributions for 3-S3 GVWs (Labelled Lines) in Manitoba: 1997 - 2018

Figures 2, 3, and 4 reveal the following observations and insights about changes in the operating GVWs of 3-S2s and 3-S3s throughout the study period.

- Figure 2 reveals an immediate (though limited) response to the increased 3-S2 GVW limit following the 1974 WCHSP. Despite a statistically insignificant change between the two distributions, the response is apparent from the rightward shift of the heavier observations in the 3-S2 GVW distributions when comparing the pre-WCHSP distribution (data collected from January to August 1974) and the post-WCHSP distribution (data collected from September to December 1974). Since no change in configuration accompanied this GVW limit increase, it appears that carriers could immediately take advantage of the increase by adding payload when hauling freight that would previously have caused the truck to weigh-out (i.e., reach its weight capacity before reaching its cubic capacity).

- As shown in Figures 2 and 3, the initial trend towards heavier 3-S2 GVWs observed in 1974 persists until *circa* 1991—three years after the RTAC MoU—when the 3-S2 GVW distribution reaches its heaviest position (i.e., the 1991 GVW distribution has the highest mean and median in the time-series dataset). This trend is consistent except for a temporary interruption in 1982, possibly owing to the regulatory changes applicable to certain configurations that occurred in that year. Nevertheless, by 1984, approximately 35% of all observed 3-S2s operated at GVWs exceeding the pre-WCHSP limit, and by 1991 this increased to approximately 50%. Figure 3 indicates the statistical significance of these GVW changes in terms of year-to-year comparisons and relative to the pre-1974 dataset. Statistically, while significant differences are evident between the 1975 and 1978 distributions and between the 1978 and 1981 distributions, it appears that the distributions observed in 1982 and 1984 are statistically similar to the 1981 distribution. No regulatory change occurred for 3-S2s during this latter period. However, every distribution from 1975 to 1991, inclusive, can be considered as taken from a population different than the one that existed prior to the 1974 WCHSP.
- Since 1991, Figures 2 and 3 reveal a downward trend in the mean 3-S2 GVW, decreasing from its maximum of 32.00 tonnes in 1991 to 25.05 tonnes in 2018. Notably, the mean 3-S2 GVW from 1992 to 2018 is lower than it was prior to the 1974 WCHSP, despite the availability of higher GVW limits. As shown in Figure 3, the distributions from 2000 to 2002 and after 2013 show no statistically significant year-to-year changes; no meaningful regulatory changes occurred during these periods for 3-S2s. Moreover, there is evidence that each distribution observed since 1991 represents a population that is different than the 1991 population. In

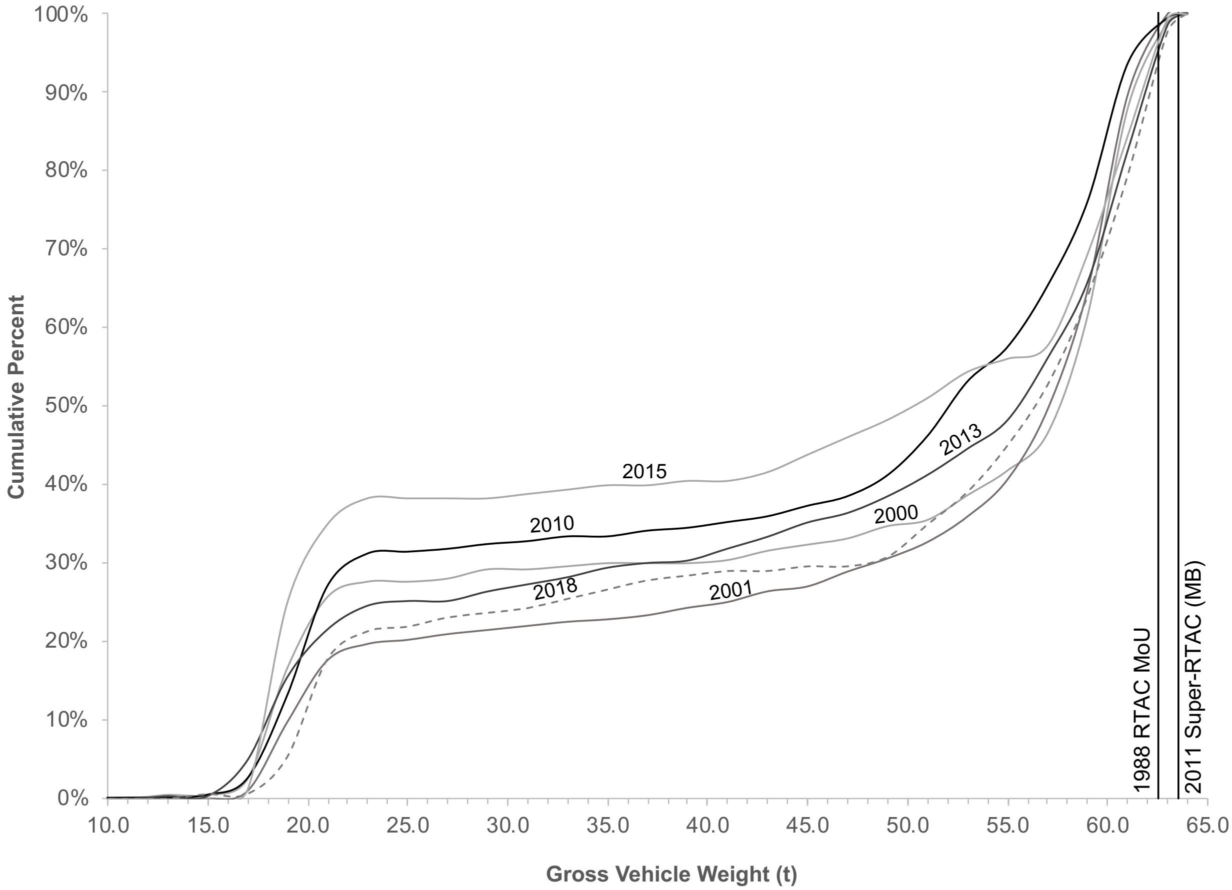
addition, Figure 2 reveals a transition toward a uniform distribution of 3-S2 GVWs across the available weight range through the study period.

- At least three factors may contribute to the downward trending mean and the shift in the shape of the 3-S2 GVW distribution. First, and of most relevance to the objectives of this article, there is evidence that, in time, carriers hauling weigh-out commodities shifted this freight to the more productive 3-S3s (or possibly eight-axle B-doubles) once these configurations became available following the RTAC MoU (see Figure 4). Consequently, the 3-S2 fleet hauled a lower proportion of higher density weigh-out freight after 1988 than prior. However, unlike the immediate increase in the GVW of 3-S2s that occurred following the WCHSP, this shift appeared to lag the regulatory change by approximately three or four years, since carriers needed to purchase new trucks to take advantage of the increased GVW limits. Second, there has been a decline in the average density of freight and the increased use of packaging, resulting in reduced payloads for the same commodity over time [33] [6]. Third, the tare weights of vehicles in some industries have decreased due to use of lighter weighted materials, such as aluminum and composites, for trailer body construction [6]. This effect is referred to as down-weighting.
- Figure 4 presents evidence of relative consistency in the upper portion of the 3-S3 GVW distribution compared to the 3-S2 GVW distribution, particularly in the last two decades. This consistency corresponds with a period in which only minimal changes to the 3-S3 GVW limit occurred. Similar to the 3-S2 distributions, there appears to be some indication of freight density reduction and down-weighting, evidenced by statistically significant changes in GVW distributions from 2001 to 2010 and 2010 to 2013. However, this effect is not consistently apparent after 2013. The 1997 3-S3 distribution shows a lower percentage of empty and lightly-loaded trucks compared to the 2000 to 2018 distributions; this may reflect the possibility that only laden vehicles were recorded during the 1997 survey. Section 6 of this article discusses the implications of this in further detail.

- The percentage of overweight observations fluctuates over the years. A lack of data about the practical application of enforcement tolerances, possible existence of permit vehicles in the dataset, and uncertainty in the level of enforcement at the time of the surveys preclude further commentary concerning overweight observations.

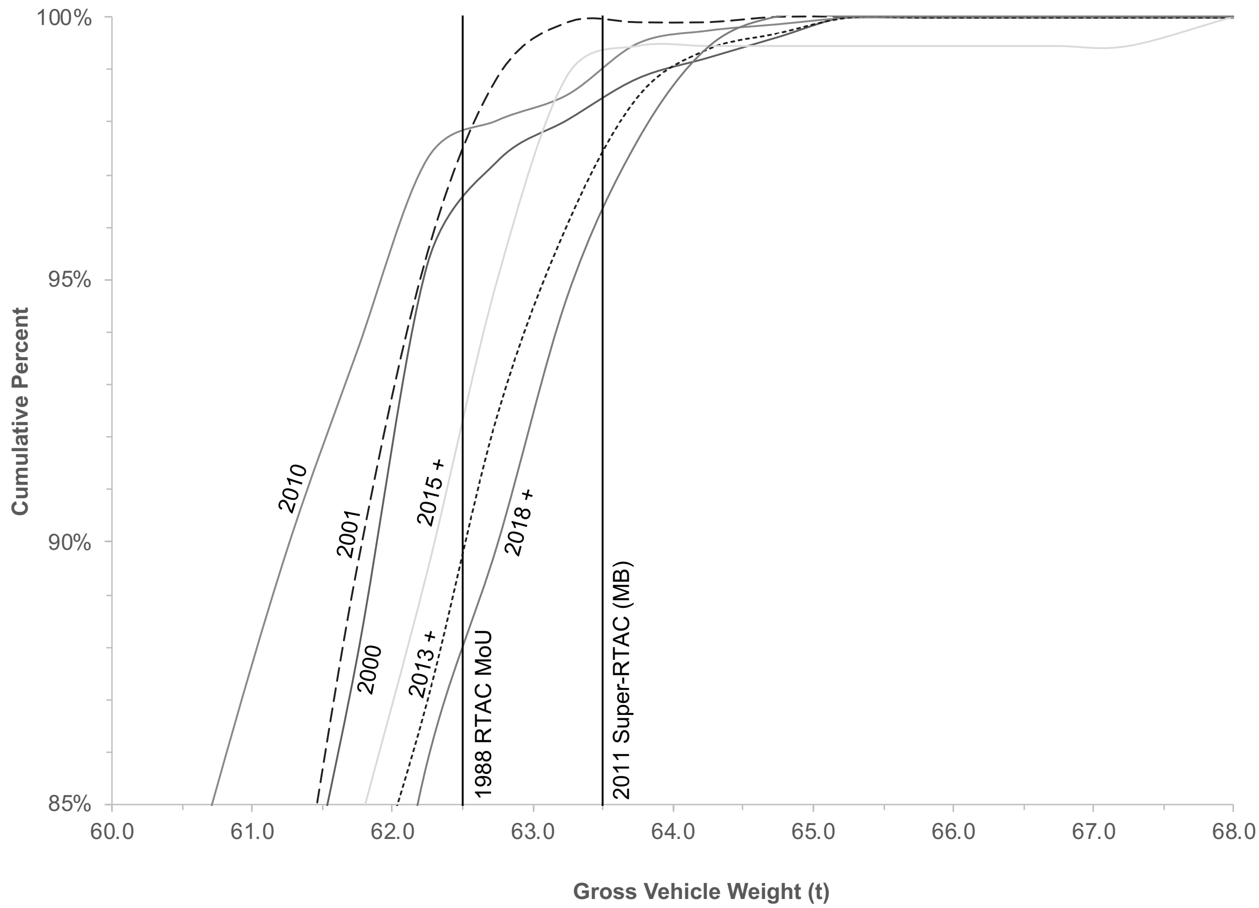
3.6.2 GVW Distributions for Eight-axle B-doubles (3-S3-S2s) and Turnpike Doubles (3-S2-4s)

Canadian carriers adopted eight-axle B-doubles after the effective introduction of the tridem axle in the 1988 RTAC MoU. Today, these vehicles comprise up to 15% of the articulated truck fleet operating on the Trans-Canada Highway [22]. Since their introduction, eight-axle B-doubles have been subject to one truck weight regulatory change in Manitoba: a 1000-kg increase in the GVW limit from 62,500 kg to 63,500 kg in 2011. Figure 5 shows the GVW cumulative frequency distributions from 2000 to 2018 and Figure 6 presents same curves magnified to display only the upper portion of the GVW distributions.



Notes: 1) the 2015 distribution relates to a survey conducted at the Headingley scale. 2) the 2018 distribution relates to a survey conducted at the West Hawk scale. 3) the remaining distributions relate to a survey conducted at both the West Hawk and Headingley scales.

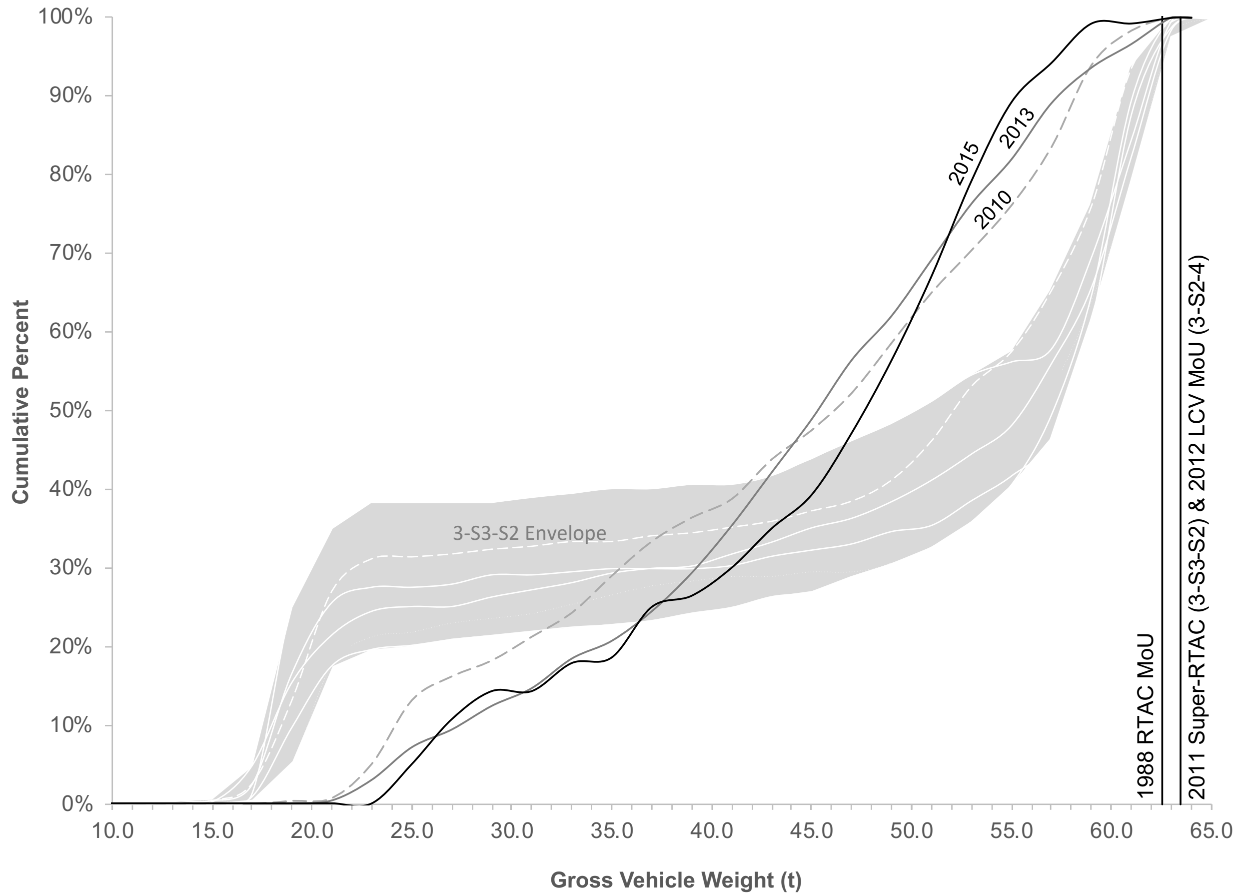
Figure 3-5: Cumulative Frequency Distributions for 3-S3-S2 GVWs in Manitoba: 2000 - 2018



Note: + indicates a significant two-sample Kolmogorov-Smirnov test at α = 0.05 relative to the 2010 distribution.

Figure 3-6: Upper Portion of Cumulative Frequency Distributions for 3-S3-S2 GVWs in Manitoba: 2000 - 2018

Due to Ontario's prohibition of LCVs on its highway network connecting to Manitoba, the Manitoba data presented for 3-S2-4 configurations has been collected at the Headingley weigh scale only and is representative of traffic on PTH 1 west of Winnipeg. Despite special permitting of turnpike doubles beginning as early as 1995, turnpike doubles only became evident in the Manitoba truck fleet mix operating on the PTH 1 after 2000 [22]. Similar to the eight-axle B-double, turnpike doubles were also subject to one 1000-kg GVW increase from 62,500 to 63,500 kg; however, this regulation change happened one year later following the LCV MoU in 2012. Figure 7 presents the cumulative GVW distributions of 3-S2-4s over the period of 2010 to 2015. To provide context, the figure includes the 3-S3-S2 GVW distribution envelope and the relevant GVW limits.



Note: The 3-S3-S2 envelope represents the maximum and minimum cumulative percent for a given gross vehicle weight for the years 2000-2018.

Figure 3-7: Cumulative Frequency Distributions for 3-S2-4 GVWs (Labelled Lines) in Manitoba: 2010 – 2015

Figures 5, 6, and 7 reveal the following observations and insights about changes in the operating GVWs of 3-S3-S2s and 3-S2-4s throughout the study period.

- As might be expected with minimal regulatory changes, both the 3-S3-S2 and 3-S2-4 distributions exhibit consistency over time, particularly in the upper portion of the distributions. There is some evidence of down-weighting and reduction in freight density for 3-S3-S2s. Limited data prior to the 3-S2-4 regulatory change precludes meaningful analysis of potential impacts.
- Observed 3-S3-S2 GVWs consistently demonstrate a bimodal distribution, with the lower peak representing empty and very lightly-loaded trucks (with a mean

around 20 tonnes) and the higher peak representing trucks operating close to the GVW limit (with a mean around 60 tonnes). The proportion of empty trucks is typically between 20 and 30% of the total, with a slightly higher percentage in 2015.

- The upper portion of the 3-S3-S2 distributions (Figure 6) reveals the industry response to the 1000-kg GVW limit increase in 2011. Statistically significant differences are evident in the upper portion of the distribution (i.e., considering GVWs greater than 60 tonnes as shown in Figure 6) when comparing the 2010 GVW distribution to the 2013, 2015, and 2018 distributions, but not when comparing the 2013, 2015 and 2018 distributions to each other. Moreover, the 2000 to 2010 distributions suggest that approximately 2 to 4% of trucks exceed the 62,500 kg limit (i.e., 96 to 98% of trucks are observed within the legal GVW limit). From 2013 to 2018 (i.e., following the increase in the GVW limit to 63,500 kg), the proportion of trucks over the previous 62,500 kg limit increased to between 8 and 12%. However, the proportion of trucks exceeding the new GVW limit remained within about 4%. The figure appears to demonstrate the industry's immediate response to the relaxed GVW limit and the consistency in the proportion of trucks that exceed the GVW limit.
- Unlike the bimodal GVW distributions for the 3-S3-S2s, the distributions for 3-S2-4s comprise GVWs across the available weight range, despite being subject to the same GVW limit. Moreover, relative to 3-S3-S2s, the 3-S2-4s exhibit little response to the increased GVW limit (from 62,500 kg to 63,500 kg), as no statistically significant differences existed between the GVW distributions. These contrasts demonstrate how the industry leverages the productivity benefits afforded by these two configurations. Specifically, 3-S3-S2s primarily haul bulk weigh-out commodities (e.g., grain, petroleum), whereas 3-S2-4s offer higher cubic capacity, ideal for hauling cube-out commodities (e.g., general freight).

3.7 LIMITATIONS

This article reported observations and insights from data collected over the past 45 years. Although this provides a unique opportunity to conduct a retrospective longitudinal study of truck GVWs, the findings should be interpreted in the context of several unavoidable, data-related limitations.

3.7.1 Location and Availability of Data

The analysis combines source data from unpublished static weigh scale surveys conducted by UMTIG at the Headingley and West Hawk weigh scales in Manitoba. As the objective of the analysis was to include as much historical data as practical, there was no attempt to control the relative sample sizes in the dataset generated from these two locations. Consequently, the analysis disregards any spatial variations in truck weights and fleet mix and the bias these differences may impose.

The analysis generally excluded truck weight data collected at the Emerson weigh scale near the United States border, since the truck weight limits applicable in the United States uniquely affect trucks operating at this location. The GVW data collected in 1974, 1975, and 1978 contains data from any primary highway in Manitoba, including data collected at Emerson. However, prior to the RTAC MoU, there was less discrepancy between the GVW limits applicable in Canada and the United States, so the inclusion of these data is not expected to materially bias the results.

3.7.2 Sampling Method

The principal sampling method limitation pertains to a change that occurred midway through the study period. Data collected from 1974 to 1984 represent laden trucks only, as the surveys conducted in those years excluded trucks with no payload. It was not specified whether the data collected between 1984 and 1998 included laden trucks or all trucks; however, based on the data it appears that only laden trucks were surveyed throughout this time as well [34]. In 1998, a modified sampling procedure designed to gather a more representative sample introduced a requirement to weigh and record data for each truck passing the weigh scale during the sampling period, including empty trucks [33].

Table 2 presents the results of a sensitivity analysis conducted to assess the potential impact of this change in the sampling method. This involved removing records with

GVWs less than an assumed tare weight and evaluating the 25th, 50th and 75th percentile GVWs compared to the original data presented for 3-S2s. Regehr et al. [35] reported mean tare weights for various 3-S2 body types in Manitoba, ranging from 13,400 kg to 15,700 kg; therefore, the assumed tare weight values for the analysis were 14,000 kg and 16,000 kg.

Table 3-2: 3-S2 GVW Statistics from the Original Data and with Presumed Empty Vehicles Removed

	1974 - Pre	1974 - Post	1975	1978	1981	1982	1984	1991	1992	1997	2000	2001	2002	2010	2013	2015	2016	2018
25th, Median, and 75th Percentile 3-S2 GVWs (kg) – Base Case Using Original Data																		
25th	27.5	27.5	27.5	27.5	26.5	25.5	25.5	28.1	24.3	23.4	21.7	21.2	20.0	20.0	18.5	17.1	16.3	18.3
Median	31.5	31.5	31.5	32.5	32.5	31.5	32.5	33.8	30.7	29.2	28.1	28.0	28.5	27.1	25.4	24.1	22.8	24.0
75th	33.5	33.5	33.5	34.5	35.5	34.5	35.5	36.1	35.4	34.6	34.6	34.6	35.2	34.4	33.9	32.1	31.9	31.3
25th, Median, and 75th Percentile 3-S2 GVWs (kg) – Assumed Tare Weight of 14,000 kg																		
	1974 - Pre	**1974 - Post**	**1975**	**1978**	**1981**	**1982**	**1984**	**1991**	**1992**	**1997**	**2000**	**2001**	**2002**	**2010**	**2013**	**2015**	**2016**	**2018**
25th	27.5	27.5	27.5	27.5	26.5	25.5	25.5	28.1	24.3	23.4	22.0	21.6	20.0	20.3	**19.4**	**20.5**	**18.7**	19.0
Median	31.5	31.5	31.5	32.5	32.5	31.5	32.5	33.8	30.7	29.2	28.1	28.1	28.7	27.2	26.0	**25.8**	**24.3**	24.4
75th	33.5	33.5	33.5	34.5	35.5	34.5	35.5	36.1	35.4	34.6	34.6	34.7	35.2	34.4	34.2	33.2	32.7	31.7
25th, Median, and 75th Percentile 3-S2 GVWs (kg) – Assumed Tare Weight of 16,000 kg																		
	1974 - Pre	**1974 - Post**	**1975**	**1978**	**1981**	**1982**	**1984**	**1991**	**1992**	**1997**	**2000**	**2001**	**2002**	**2010**	**2013**	**2015**	**2016**	**2018**
25th	27.5	27.5	27.5	27.5	26.5	25.5	26.5	28.1	24.5	23.7	**23.8**	**23.5**	**21.7**	**22.2**	**21.5**	**22.0**	**21.5***	**21.1**
Median	31.5	31.5	31.5	32.5	32.5	31.5	32.5	33.8	30.8	29.3	29.0	29.1	**30.0**	**28.6**	**27.4**	**26.8**	**25.9***	**25.7**
75th	33.5	33.5	33.5	34.5	35.5	34.5	35.5	36.1	35.4	34.7	35.0	35.0	35.4	34.8	34.7	**33.8***	33.4	32.6

Notes:
Bold cells denote changes in gross vehicle weight percentile greater than 5%.
* The star (*) indicates the year and tare weight with the maximum percent change.

Prior to 2000, the 25th, median, and 75th percentile 3-S2 GVWs are insensitive to the removal of presumed empty vehicle records, regardless of the tare weight assumption. From 2000 on, the most significant change resulting from the removal of assumed empty vehicles occurs for the 25th percentile values. This effect intensifies as the assumed tare weight increases, with a maximum 32% difference observed for the 25th percentile 3-S2 GVW in 2016. By comparison, the median and 75th percentile GVWs are less sensitive, with maximum differences of 14% and 5% observed in 2016 and 2015, respectively. Because the principal findings related to these data pertain to the heavier observations within the sample, the observations and insights presented are considered relevant despite the change in the sampling method over time.

3.7.3 Other Limitations

Finally, limitations surrounding the representativeness of the results due to the enforcement intensity at the time of the surveys could not be quantified. Previous research indicates that high levels of enforcement lead to near complete compliance [33] and changing enforcement levels throughout the study period may have impacted the GVW trends. However, this impact was considered minor and an analysis of enforcement levels was outside the scope of this article.

3.8 CONCLUSIONS

Three primary truck size and weight policy changes occurred in Canada over the past five decades: the 1974 WCHSP, the 1988 RTAC MoU, and special permitting of LCVs. These regulatory changes introduced new truck configurations and influenced the GVWs of trucks operating on principal highways. This retrospective longitudinal study contributed unique *ex post* evidence of these changes for 3-S2s, 3-S3s, 3-S3-S2s, and 3-S2-4s using a time-series of weigh scale data collected on the Trans-Canada Highway in Manitoba, Canada. In particular, it sought to answer three questions: *How do operating GVW distributions change when axle and GVW limits are relaxed? How do these changes differ for predominant truck configurations? How long does it take for the trucking industry to respond to truck weight regulatory changes?*

The analysis revealed the following principal conclusions:

- As a direct result of purposeful truck regulatory changes in Canada, trucks operating on many primary Canadian highways carry heavier and larger payloads today than they did five decades ago. Carriers adapted to regulatory change by adopting new configurations, diversifying their fleets, and when possible and advantageous, increasing the operating GVWs of their trucks. Likewise, periods of minimal regulatory change corresponded to relative stability of operating GVWs.
- Certain configurations subject to effectively the same GVW limits exhibited markedly different GVW distributions and contrasting responses to regulatory change, depending on whether they typically haul weigh-out or cube-out commodities. Most notably, compared to 3-S2-4s, 3-S3-S2s featured bimodal distributions (i.e., either empty or fully-loaded) rather than uniform GVW distributions, and had an observable sensitivity to increases in the GVW limit. These contrasts demonstrated how the industry leveraged the productivity benefits (for weight and cube) facilitated by a diverse fleet mix.
- Increasing the GVW limit for a particular truck configuration led to immediate increases in operating GVWs for the portion of those configurations used to haul weigh-out commodities. This effect was apparent for 3-S2s following the WCHSP in 1974, for 3-S2s following the RTAC MoU in 1988, and for 3-S3-S2s following the 1000-kg increase in GVW in 2011. The timeliness of this response appeared contingent on the immediate availability of the configuration.
- When regulations introduced a new, more productive configuration, the industry's response lagged the regulatory change by approximately three or four years as carriers adjusted their fleets and logistics practices. The increasing 3-S2 GVWs between 1988 and 1991 provided evidence of this transition period and of the ongoing downward effect that 3-S3s had on the 3-S2 GVWs since 1991.
- Two exogenous trends—namely the ongoing reduction in average freight density and the down-weighting of truck tare weight—appeared to influence the operating GVWs of all four truck configurations examined.

Despite the noted data-related limitations and the constrained geographic scope of the data collected, the evidence presented in the article contributes new insights that help address gaps and persistent modeling uncertainties applicable to the prediction and evaluation of the impacts of truck weight regulatory changes. In particular, the findings reveal how regulatory changes differentially influenced truck operating GVWs for an evolving and diverse truck fleet. Further research would be needed to compare the impacts theoretically predicted using an *ex ante* modelling approach to the observations presented herein.

3.9 ACKNOWLEDGEMENTS

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4.0 **Reliability Analysis of Typical Highway Bridges in Manitoba Designed to the Modified HSS-25 Live Load Model**

Nationally specified TSW regulations establish the maximum allowable truck size and weight limits for major highways in Canada; however, individual provinces can adopt heavier weight limits to accommodate for locally operating permit vehicles and industry specific live loads. To account for the increased force effects on bridge infrastructure, localized modifications to the bridge design code live load model also follow to ensure that bridge level of safety is maintained. This paper presents a case study that determines the reliability of three typical bridge types in Manitoba subject to historic truck operating weights and designed in accordance with an uncalibrated local live load model specified by Manitoba Infrastructure. This chapter addresses the final research question: How has bridge reliability been impacted by changes in live load models and truck operating weights in Manitoba?

The material in this chapter was submitted for publication in the Canadian Journal of Civil Engineering on December 21, 2020 and is printed here with permission of co-authors Jonathan Regehr, Graziano Fiorillo, Aftab Mufti and Basheer Algohi. The paper is self-contained in the chapter with its own abstract, introduction, conclusion and references. The author of this thesis was the lead author, researcher and shared analyst and had principal responsibility for all aspects of the paper, while co-authors provided advice, portions of the analysis and reviews. A sample reliability index calculation for a steel bridge with a span length of 24 m is provided in Appendix A. Appendix B presents the tabulated mean load and resistance values for moment and shear that were used for the reliability analysis.

4.1 ABSTRACT

Several provinces in Canada have modified the live load model specified in national bridge design codes to account for locally permitted truck types and industry specific loads during bridge design and evaluation. Manitoba similarly introduced a live load model for the design of provincial bridges in accordance with AASHTO LRFD, referred to as the Modified HSS-25. This article presents truck weight datasets and methods used to develop Manitoba-specific live load statistics to conduct a reliability analysis for

three typical simply supported structure types: precast prestressed concrete box girder, precast prestressed concrete I-girder and steel girder. Reliability analyses were conducted to consider the live load statistics established for the calibration of the AASHTO LRFD code and the local live load statistics, to compare with the target reliability index of 3.5. The analyses determined that average reliability indices ranged from 4.69 to 4.95 with respect to the AASHTO LRFD live load statistics and 4.65 to 5.04 for the considered span lengths and structure types with respect to the Manitoba statistics. These results demonstrate a level of safety that exceeds the code requirements, indicating that structures designed in accordance with the HSS-25 potentially possess the structural capacity to withstand increased vehicular load effects for the considered bridge types and span lengths.

4.2 INTRODUCTION

Canadian truck size and weight regulations specify the maximum allowable truck loads, and in accordance with this, bridge design code live load models have been developed and calibrated to account for the wide range of trucks on the highway network. Despite the relatively uniform regulations across the country, individual provinces are able to increase their allowable truck size and weight policies and consequently adapt their bridge design live load model to account for locally specific truck types (e.g., mobile cranes and routinely used permit vehicles) and loading requirements. Examples of this include British Columbia and its specification of the BCL-625 truck model, Alberta and its specification of the CL-800 truck model, and Ontario and its specification of the CL-625-ONT truck model (Alberta Transportation, 2018; British Columbia Ministry of Transportation and Infrastructure, 2016; CSA, 2019).

Each of these models provides minor modifications to design truck axle weights or spacings and increases the design loads beyond the minimum specified Canadian Highway Bridge Design Code (CHBDC) live load model, the CL-625. For example, the BCL-625 truck model was developed following analytical comparisons, which included load ratings of existing structures subject to British Columbia's current truck fleet. The CL-625 load model was found to be insufficient on short spans for cranes and on medium length continuous spans for 85-tonne permit vehicles, resulting in the modified design live load (British Columbia Ministry of Transportation and Infrastructure, 2016). Similarly, the CL-800 load model was selected by Alberta Transportation after a detailed comparison of the previous design load, the CS-750 (according to CSA S6-88), following the release of CHBDC in 2000 (UMA Engineering Ltd., 2001). This study ensured that the capacity of bridges would not limit the ability of the Alberta highway system to carry overloads and also allowed the opportunity for some truck weight limit increases in the future.

Although Manitoba Infrastructure (MI) specifies bridge design according to the American Association of State Highway and Transportation Officials (AASHTO) load and resistance factored design (LRFD) bridge design code and not CHBDC, Manitoba has similarly specified a unique truck for the design of provincial bridges, referred to as the Modified HSS live load model (Manitoba Infrastructure and Transportation, 2011). However, despite the use of this live load model for several decades, no formalized analyses have been performed to assess the resulting reliability indices for typical Manitoba bridge structures.

The purpose of this article is to determine the level of safety resulting from the use of the uncalibrated HSS-25 live load model with the AASHTO LRFD bridge design code safety factors for the design of bridges in Manitoba. There are two primary objectives of this work:

1. to present the case study approach and database of Manitoba-specific truck weights and spacings that were observed over a two-decade period; and
2. to conduct a reliability analysis for three typical bridge types in Manitoba with respect to the AASHTO LRFD live load statistics that were used to calibrate the AASHTO code and with respect to statistics developed from truck data in Manitoba.

Existing bridges could also be evaluated to determine their reliability index based on the actual resistance determined from load tests; however, the focus of this article is on the design live load model for new bridges assumed to be designed to satisfy the requirements of the factored loads, as discussed in the methodology section.

The remainder of this article is structured as follows. Section 2 presents a brief background of the calibration procedures for AASHTO LRFD and the Manitoba bridge design specifications. Section 3 describes the approach, data sources and methodology for the reliability analysis. Section 4 presents and discusses the resulting reliability indices. Section 5 outlines limitations of the analysis. The last section summarizes key contributions and presents concluding remarks.

4.3 BACKGROUND

Structural performance can be measured in terms of reliability or probability of failure, and LRFD bridge codes are developed and calibrated to meet a predefined target level of safety. For LRFD design codes, the general design equation is expressed as

$$\sum_i \boldsymbol{\alpha_i \cdot X_i < \phi R_n}$$

Equation 4-1

where α = load factor for component i; X_i = nominal design load for component i; ϕ = resistance factor; and R_n is the nominal design resistance (Nowak, 1995).

The purpose of reliability-based code calibration is to determine the required load and resistance factors such that the safety of bridges designed according to a specific code will have a consistent and uniform level of safety regardless of bridge type and span length. This process accounts for the statistical variability in both the load and resistance models, resulting from uncertainties in parameters such as material strength, member geometry and dimensions, analysis procedure simplifications and the actual loads of truck traffic. Once the load and resistance models are developed and a target reliability index is determined, a set of load factors are calibrated so that each load component has on the average the same probability of being exceeded. The resistance factors are calculated such that the structural reliability is close to the target reliability index (Nowak, 1995).

Following these principles, the first edition of the AASHTO LRFD Bridge Design Specifications was published in 1994. This code was based on an extensive study that developed load and resistance models, applied a reliability analysis procedure and calculated reliability indices for various bridge types and span lengths (Nowak, 1999). Through this process, the design live load model, the HL-93 and associated tandem and lane loads, as shown in Figure 1, were developed to replace the historic HS-20 load model and more adequately envelope the range of truck loads that a typical bridge structure might experience on the United States interstate highway system (Kulicki & Mertz, 2006; Kulicki & Stuffle, 2006).

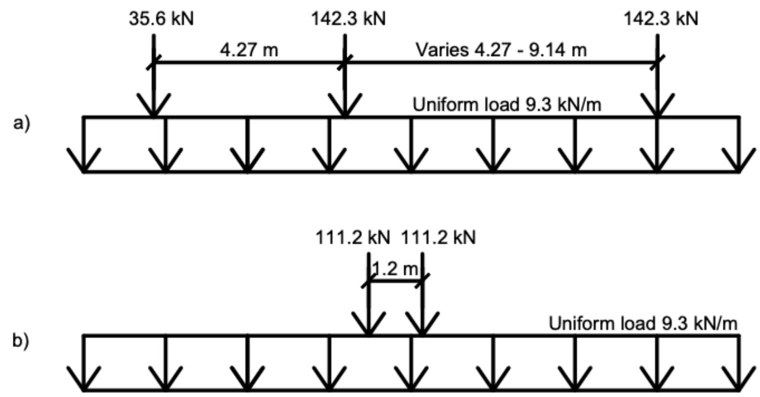


Figure 4-1: Live load models for: (a) HL-93 standard design truck and lane load, and (b) design tandem and lane load (AASHTO, 2020)

The live load statistics used for calibration of the LRFD code were developed from a truck weight survey conducted in the province of Ontario in 1975. During calibration, the load effects were extrapolated to represent the expected 75-year bridge life moment and shear effects. Further explanation of the code calibration and live load extrapolation procedures are documented elsewhere (Kulicki et al., 2007; Nowak, 1993; Nowak, 1995; Nowak, 1999).

Upon conclusion of the AASHTO LRFD code calibration, the basic load combination for determining the ultimate capacity of members, or design equation, was determined as follows:

$$R_n = [1.25(DC) + 1.5(DW) + 1.75(LL \times IM)] / \phi$$

Equation 4-2

where DC is the dead load of structural components and non-structural attachments; DW is the dead load of the wearing surface and utilities; LL is the load effect resulting from the calibrated vehicular live load model (i.e., the HL-93 or HSS-25); IM is the dynamic load allowance, equal to 1.33; and ϕ is the resistance factor for each structural component reflecting its use (AASHTO, 2020).

Unlike most other provinces in Canada, Manitoba does not design its provincial highway bridges in accordance with CHBDC and instead specifies the AASHTO code with an amended Structures Design Manual (Manitoba Infrastructure and Transportation, 2011). In the 1980s, it was recognized that heavier trucks, including double trailer combinations subject to evolving Canadian truck size and weight limits, operated in Manitoba with load effects potentially exceeding those produced from the AASHTO Standard Specifications HS-20 load model. Consequently, MI established a new live load model, the Modified HSS-25, to use in addition to the AASHTO design load models. Current MI bridge engineers noted that this load model was originally developed by adapting the AASHTO HS-20 model, increasing it by 25% (i.e., 25/20) and adding an additional trailer with the same inter-axle spacing, as shown in Figure 2 (A. Pankratz, personal communication, 2020). This model is not used in conjunction with a lane load.

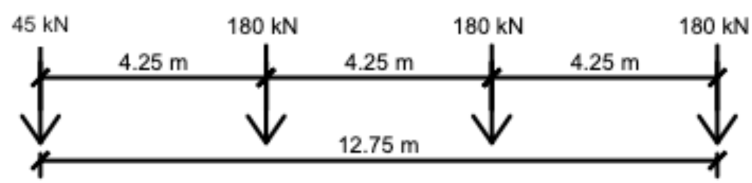


Figure 4-2: Modified AASHTO HSS-25 Truck (Manitoba Infrastructure and Transportation, 2011)

Following release of the AASHTO LRFD code in 1994, the HSS-25 live load model continued to be specified for highway bridge designs in Manitoba, in addition to the HL-93 model, using the load and resistance factors in Equation 2 that were calibrated for the HL-93 model. Use of this live load model for design has been found by MI to be beneficial when evaluating existing structures for permit or overload vehicles (A. Pankratz, personal communication, 2020). However, formal studies have not yet been completed to evaluate the safety margin of the HSS-25 live load model for bridge design with the provided load and resistance factors specified in AASHTO.

This article addresses this research need by conducting reliability analyses to determine the reliability of representative Manitoba bridges designed using the HSS-25 live load model. The approach will consider the live load statistics used for the calibration of the AASHTO LRFD code and the live load statistics extracted from Manitoba-specific truck weight data. Notably, MI has recently started to design bridges on heavy haul routes using the HSS-30 truck, which has the same configuration as the HSS-25 but increases the axle weights by 50% (i.e., 30/20) compared to the HS-20 load model. These increases were specified in an effort to account for possible heavy permit or overloaded trucks, especially for highways connected to the oil industry in Western Manitoba (A. Pankratz, personal correspondence, 2020). The focus of this article is on typical truck weights in Manitoba and as a result, only the HSS-25 is addressed in the following sections.

4.4 METHODOLOGY

This section presents the case study approach, data sources and the step-by-step method used to conduct the reliability analysis of three typical bridge types designed in accordance with the AASHTO LRFD design code and the Modified HSS-25 live load model. Projected live load statistics developed from truck data collected in Manitoba are obtained by applying the L-moments method of the extreme value theory on the distribution of the daily maximum load effects on bridges (Stedinger et al., 1993). The load statistics related to the HL-93 load effect are those from the calibration of the AASHTO LRFD bridge design standards (Kulicki et al., 2007; Nowak, 1999). The nominal resistance of the bridge members is computed according to AASHTO LRFD using a sample of ten bridges built in Manitoba and designed to the Modified HSS live load model. Finally, the reliability analysis of bridge members is conducted using the first-order reliability method (FORM) (Nowak & Collins, 2000).

4.4.1 Case Study Approach

The Manitoba bridge inventory was referenced to determine representative bridge types and span lengths for the case study reliability analysis. Using this database, the structures were selected by filtering the inventory by (1) location and roadway classification, (2) bridge type, (3) span length, and (4) number of spans. The following paragraphs outline the selection process.

The inventory records details regarding the location and geometric properties of all 3,647 water control structures under MI's jurisdiction, including culverts and bridge types not found on provincial trunk highways (PTHs), such as provincial roads and farm/field accesses. As a result, the bridge location data and road classifications were filtered such that only PTH routes were considered. To ensure alignment between the locations of the bridges analyzed and the sources of available truck data (all of which was collected on PTH 1—the Trans-Canada Highway), bridge types along PTH 1 were considered to be the most representative structure types.

The frequencies of the various structure types were used to further refine the bridge selection. The three representative bridge types selected for the reliability analysis include a precast prestressed concrete box girder bridge, a precast prestressed concrete I-girder bridge, and a steel girder bridge. These structure types comprise 13%, 30% and 37% of the structure types on PTH 1 and 12%, 16% and 19% of all PTH bridge structures in the MI inventory. Although timber bridges comprise 61% of the MI bridge inventory and 27% of PTH bridges, 0% of the bridges on PTH 1 were timber. For this reason, timber bridges were not considered as a structure type that would directly experience the load effects of the observed trucks on PTH 1.

The selected span length range and number of spans considered for the case study analysis were again based on the bridge inventory. The average maximum span length for concrete box girder bridges on PTH 1 was 26 m and 25 m for all PTH bridges of this type. Similarly, the average maximum span length for concrete I-girder bridges on PTH 1 was 21 m and 22 m for all PTH bridges, and 18 m and 28 m for steel girder bridges. Based on the average span lengths and representative drawings for structures designed to AASHTO LRFD and the HSS-25 that were obtained from MI, structures with spans ranging from 16 to 26 m were considered for the reliability analysis.

The bridge drawing dataset contained three steel girder bridges, three precast prestressed concrete box girder bridges and four precast prestressed concrete I-girder bridges. All the precast prestressed concrete box and I-girder bridge drawing sets provided by MI had simply supported spans, even for multi-span structures. Of the three steel girders analyzed, one was simply supported and the other two were three-span continuous with an identical girder cross section, despite having different intermediate span lengths. The matching girder cross section for two different intermediate span lengths precluded the definition of a regression model to describe the self-weight of the cross sections at the supports as a function of span length for negative bending; therefore, the nominal resistance of the cross sections for negative bending could not be defined using this dataset. For positive bending, the effective length of the main continuous span was used for the evaluation of the nominal capacity of the three steel girder structures. Only simply supported structures (positive moment and shear load effects) were considered for the reliability analysis.

4.4.2 Truck Data Sources and Screening

The data presented in this study includes unpublished on-road truck surveys conducted from 2000 to 2018 by the University of Manitoba Transport Information Group (UMTIG) at the Headingley and West Hawk permanent static weigh scales. As shown in Figure 3, the Headingley weigh scale is located approximately 7.5 km west of Winnipeg on PTH 1. This scale processes interprovincial and intra-provincial truck traffic. The West Hawk weigh scale is located approximately 1.2 km west of the Manitoba-Ontario border on PTH 1. Due to its close proximity to the border, the majority of traffic moving through this scale is interprovincial and trucks must comply with both the Manitoba and Ontario truck size and weight regulations. A permanent, static weigh scale is also located in Emerson; however, this article focuses on the truck traffic movements on PTH 1. The static weigh scale datasets do not provide axle spacing information. To obtain the axle spacing data, two weigh-in-motion (WIM) stations on PTH 1 were used in this paper to collect data on a per vehicle basis: Station 65 west of Winnipeg and Station 61 east of Winnipeg, as shown in Figure 3.

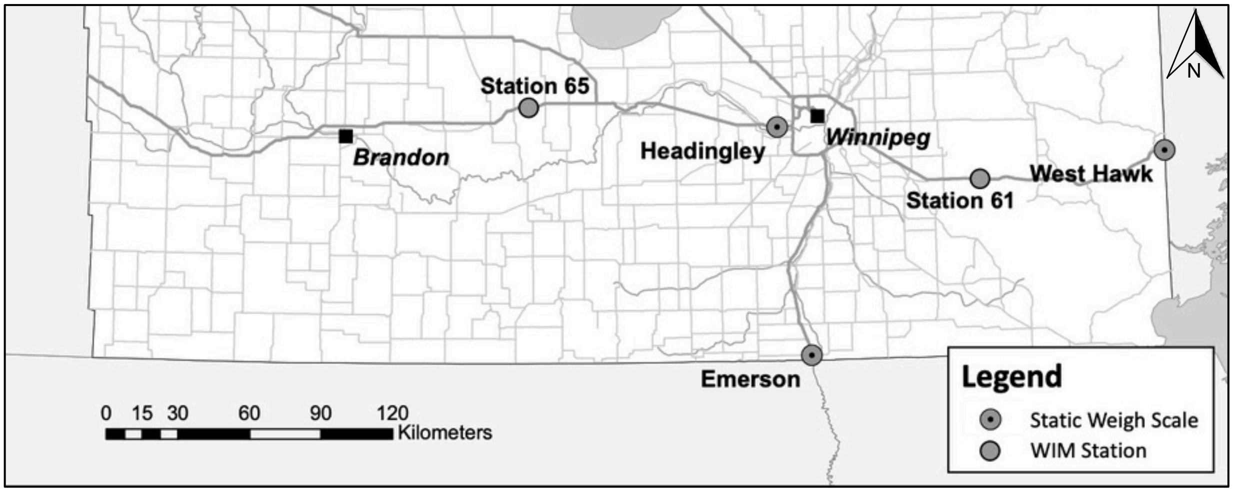
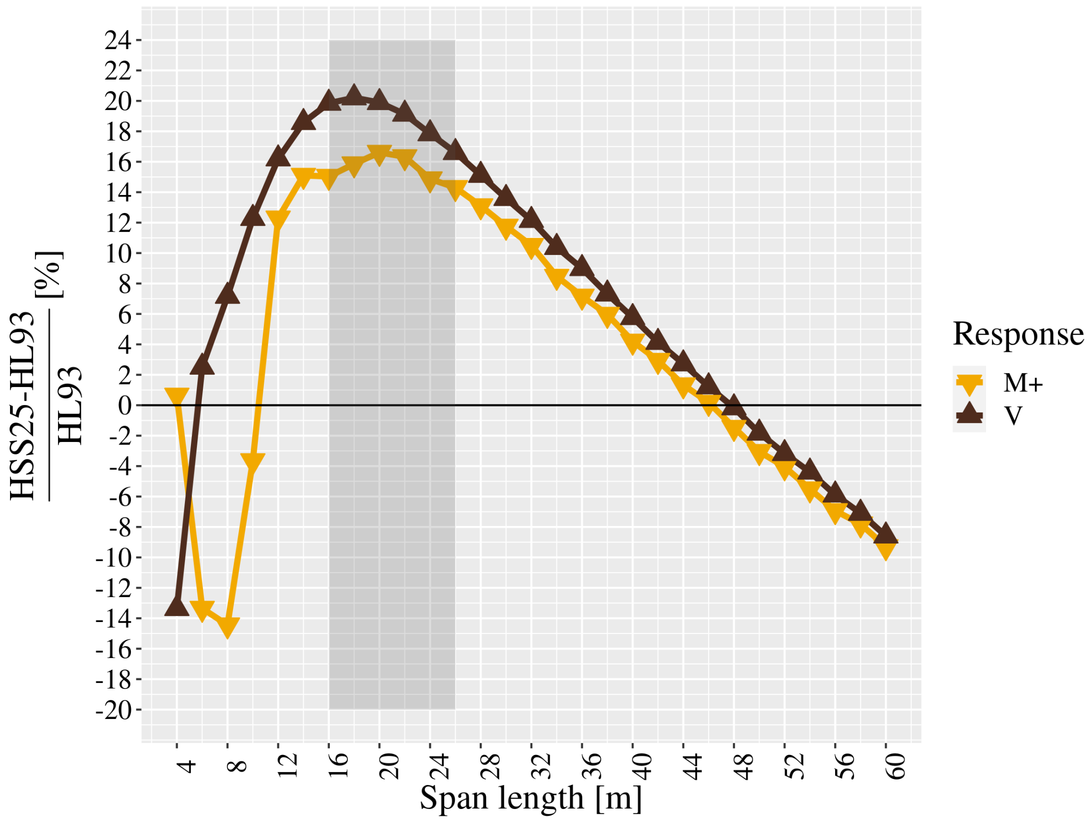


Figure 4-3: Static Weigh Scale and WIM Station Locations in Manitoba, Canada

The static weigh scale dataset includes manual classification of trucks and the axle group weight measurements. For the following analysis, only five-axle semitrailers (3-S2s), six-axle semitrailers (3-S3s) and B-doubles (3-S3-S2s) were considered; the dataset contains a total of 20,929 trucks observed at the two weigh scales on PTH 1. These three configurations have comprised 85 to 95% of the truck fleet operating on the Trans-Canada highway in Manitoba over the past two decades, based on weigh scale data (Di Cristoforo et al, 2012; Wood & Regehr, 2017). The remaining percentage of the fleet primarily includes straight trucks and longer combination vehicles (LCVs). As shown in Figure 4, for the selected simply supported span length range, the positive moment and shear load effects produced by long hauling trucks (represented by the HSS-25) are greater than those produced by single unit short trucks (represented by the HL-93 truck) in the span range evaluated. Therefore, the effect of straight trucks was not considered for the projection of the extreme statistics of the truck response. LCVs were also not considered in the analysis as they tend to produce less severe ultimate load effects than the B-double for simply supported spans, as are analyzed in this article.



Note: grey shaded area represents the span range considered in the analysis.

Figure 4-4: Percent difference between positive moment and shear response for simply supported structures subjected to the HSS-25 live load model and the HL-93 live load model

The weigh scale dataset examined for the reliability analysis was screened to remove any invalid results. Entries with the incorrect number of axles for the specified configuration were removed from the dataset. Following the same criteria used in previous studies conducted at the University of Manitoba with similar datasets, axle group weights from the static weigh scale surveys that were less than 1,500 kg or more than 150% of the allowable axle limit were deleted from the analysis and were considered erroneous (Tan, 2002). Only entries with valid axle group weights for every axle were considered in the structural analysis. It was assumed for the analysis that the weight per axle group was equally distributed to each axle in the group.

Two sets of annual bidirectional WIM data were analyzed to determine axle spacings, one from 2006 and the other from 2017. This was done to evaluate the consistency in axle spacing in time for the three selected truck configurations observed at these sites.

The raw WIM data was processed to include only those records that could be considered 3-S2, 3-S3 or 3-S3-S2 configurations. This included removal of entries with the incorrect number of axles or vehicles that were more than 10% beyond the maximum allowable length for the truck configuration considered. Following this, the data was cleaned and spacings that did not meet interaxle spacing regulatory requirements for the particular truck type (as set out by the Council of Ministers of Transportation and Highway Safety, 2019) were also removed from the dataset. After the data was cleaned, the weighted average interaxle and axle group spacing from the two WIM stations was determined for each truck type and was rounded to the nearest 100 mm. It was assumed that the spacing between axles in a tridem were equal. Table 1 lists the axle spacings assigned to the vehicle configurations.

Table 4-1: Weighted average axle spacings [m] by truck configuration in Manitoba (WIM data from 2006 and 2017)

	Spacing between Axles 1 & 2	Spacing between Axles 2 & 3	Spacing between Axles 3 & 4	Spacing between Axles 4 & 5	Spacing between Axles 5 & 6	Spacing between Axles 6 & 7	Spacing between Axles 7 & 8
3-S2	5.10	1.30	10.50	1.30	-	-	-
3-S3	5.10	1.30	9.30	1.65*	1.65*	-	-
3-S3-S2	5.10	1.40	6.30	1.55*	1.55*	5.70	1.30

Note: *equal spacings were assigned to axles within a tridem group

4.4.3 Analytical Method

The level of safety of three typical Manitoba bridge types designed according to the HSS-25 live load model within the specified span length range can be assessed using the reliability analysis method, as described below. Details regarding the development of the reliability analysis method and the statistical databases used for the AASHTO LRFD bridge code are provided in other references (Kulicki et al., 2007; Nowak, 1993; Nowak, 1994; Nowak, 1995; Nowak, 1999).

4.4.3.1 Live Load Model Based on Manitoba Live Load Data

The static weigh scale and WIM databases were used to develop Manitoba-specific live load effects. Influence line analysis was conducted by passing each truck in the two-decade database over the specified span lengths to determine the shear and moment effects for simply supported bridge spans. A frequency analysis was performed to fit the upper tails of the daily maximum load effects to a normal distribution (Stedinger et al., 1993). A normal distribution well describes the upper tails of the daily maxima bridge responses as indicated by the R^2 coefficient shown in Table 2 for several span lengths. The table also shows the mean and the coefficient of variation (COV) of the normal distributions.

Table 4-2: Statistical parameters of daily maximum moment and shear load effects from Manitoba truck data (2000 – 2018)

Span [m]	Bending			Shear		
	Mean [kN.m]	COV	R^2	Mean [kN]	COV	R^2
16	927.0	0.028	0.921	277.4	0.025	0.925
18	1125.9	0.026	0.915	299.9	0.022	0.912
20	1363.5	0.027	0.888	325.7	0.019	0.966
22	1654.1	0.025	0.891	350.9	0.019	0.959
24	1933.9	0.024	0.894	369.8	0.018	0.953
26	2224.6	0.023	0.895	390.5	0.018	0.945

Due to the limited annual sample sizes, a representative estimate of the standard deviation of load responses could not be obtained and the COV of the response was quite low, around 2% on average as shown in Table 2. This low value resulted because the aforementioned COV only accounted for the variability of the load effect, and neglected other important uncertainties such as geographic variability, data errors, dynamic impact and load distribution. A comparison of several live load models by Ghosn et al. (2011) showed that typical COVs of live load responses are in the order of 0.15 to 0.26, which is in agreement with the live load COV of 0.18 used for the calibration of the AASHTO LRFD (including dynamic impact) (Kulicki et al., 2007). For this study, a COV equal to 0.15 was selected to account for these uncertainties without the effect of dynamic impact. For the dynamic impact effect, a bias factor of 1.10 was used, as was assumed during the initial AASHTO LRFD calibration (Nowak, 1999).

The influence line analysis and resulting mean daily maximum load effects per year also revealed that in Manitoba the average truck effect on bridges has been increasing since 2000, as shown in Figure 5. The average growth rate during this period is approximately 0.35% per year over the past 18 years. This effect is likely influenced by an allowable truck weight increase in 2011; the reduction in load effects in 2016 may have been a result of the reduced truck weight collection sample size in that year relative to other years.

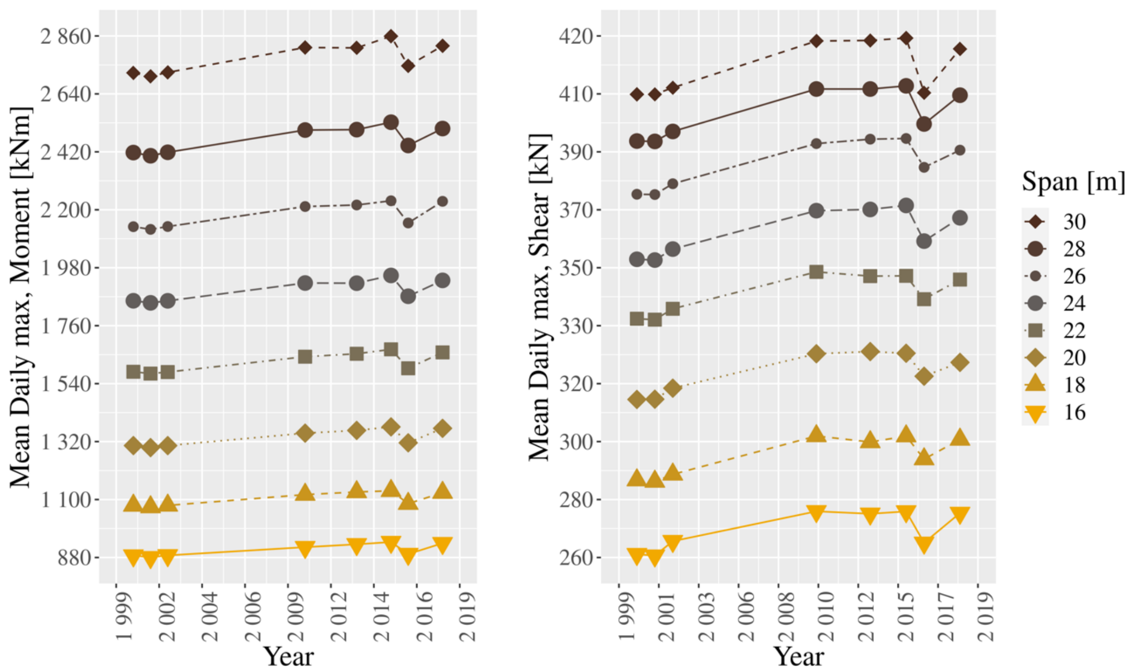


Figure 4-5: Mean daily maximum moment and shear load effects per year on PTH 1 in Manitoba (2000 – 2018)

To obtain a statistical projection of the daily maximum load effects over the design life of bridges, the initial normal variate of the daily responses corresponding the mean value of the upper tail for each bridge span, whose parameters are listed in Table 2, was projected into the future for N_y years to account for the number of events expected in this time frame by means of the extreme value theory (Ang & Tang, 1984). Assuming that the initial variate of the daily maximum load effect is time invariant, then the cumulative distribution at time t, $F_t(x)$, can be obtained using the following equation:

$$F_t(x) = F_i(x)^{Nev}$$

Equation 4-3

where $F_i(x)$ is the cumulative distribution of the initial variate of the response x (either in bending or shear); and N_{ev} is the total number of number of events throughout the number of years, N_y. In this case study, the total number of events is N_{ev} = 365 days × 50 years. N_y was set equal to 50 and not 75 years to allow for comparison to the AASHTO LRFD model, which was developed about 25 years ago and extrapolated loads for a bridge design life of 75 years.

Another consideration is that the statistics of the daily maximum load effects listed in Table 2 are for a single lane load, while the base case design scenario in AASHTO LRFD is for two-lane loading. The available truck data was not detailed enough to describe the simultaneous occurrence of multiple lane effects. To overcome this difficulty, a convolution integral was used to obtain the bridge response for two loaded lanes (Ghosn et al., 2011):

$$f_i\,(x) = \int_{-\infty}^{\infty} f_2\,(x - z) \cdot f_1(z)\, dz$$

Equation 4-4

where $f_i(x)$ is the probability density function of the convolution for the response x, while $f_1(...)$ and $f_2(...)$ are the probability density function of the daily maxima effect on lanes 1 and 2, respectively. In these calculations it is assumed that the distributions of the daily maxima on lanes 1 and 2 are the same.

In AASHTO, the response of the live load on bridges is transferred to the most affected member by means of a simplified distribution factor (DF) equation, which includes the multiple presence effect of the two-lane loading (AASHTO, 2020). To redistribute the effect of the convolution integral for the Manitoba load response to the member equivalent effect, the DF was divided by 2 to remove the multiple presence effect embedded into the AASHTO DF equations for two lanes (Ghosn et al., 2011). In the calibration of the code, it was assumed that the DF corresponded to the actual mean value of the DF (i.e., a bias factor = 1.0) for converting the two lane load to the per girder effect in the reliability analysis (Kulicki et al., 2007). The same approach is used in this case study.

Following this method, the projected cumulative distribution of the daily maximum load effect at time t, Ft(x) (Equation 3), was found to resemble a Gumbel Type I extreme value distribution (Ang & Tang, 1984).

4.4.3.2 Live Load Model Considered in AASHTO LRFD Calibration

To compare the difference in reliability indices resulting from the Manitoba live load statistics to the ones resulting from the live load statistics developed during the AASHTO LRFD code calibration, live load parameters from the previous 1975 Ontario truck data were referenced. The live load statistical model for two-lane loading used for the development of AASHTO LRFD consisted of a lognormal distribution with a mean value expressed by the product of the nominal HL-93 load response and a bias factor (Kulicki et al., 2007). The final bias factors for two lanes loaded in the calibration report ranged from 1.10 to 1.20 for positive bending and 1.08 to 1.18 for shear and were based on an average daily truck traffic volume of 5000 (Kulick et al., 2007). A bias factor of 1.18 was used for both bending and shear in the presented reliability analysis. The COV of the live load distribution was 0.12 without considering the dynamic effect. For the dynamic impact effect, a bias factor of 1.10 and a combined COV for live load and dynamic allowance of 0.18 were considered (Kulicki et al., 2007; Nowak, 1999).

4.4.3.3 Dead Load Model

The dead load parameters were calculated for the sample of ten representative bridges provided by MI based on the cross-sectional geometry extracted from the drawings. From these sample bridges, linear regressions for the self-weight of girders and the superimposed dead loads (DL_1 and DL_2), consisting of factory-made materials and cast-in-place materials, and the weight of the wearing surface (DW) were established as a function of the span length. The regression parameters for the dead load components are listed in Table 3. These parameters allowed for estimation of the dead load effects for different span lengths within the specified range.

Table 4-3: Dead load regression parameters for structures designed according to the HSS-25 live load model as a function of span length

Bridge Type	Factory-Made Materials DL_1			Cast-in-Place Materials DL_2			Wearing Surface DW		
	Intercept [kN/m]	Slope [kN/m^2]	R^2	Intercept [kN/m]	Slope [kN/m^2]	R^2	Intercept [kN/m]	Slope [kN/m^2]	R^2
Precast Prestressed Concrete Box Girder	3.6	0.50	0.947	-10.6	0.61	0.838	2.6	-	-
Precast Prestressed Concrete I-Girder	10.1	0.03	0.028	10.5	0.06	0.088	5.3	-	-
Steel Girder	1.7	0.08	0.840	17.3	0.07	0.078	5.3	-	-

It should be noted that the low value of the R^2 coefficient for the parameters of DL_1 and DL_2 for precast prestressed concrete I-girders and steel girders is due to the small variability of the cross sections within the analyzed span range. The average thickness of the wearing surface recorded on the sample drawing sets was found equal to 85 mm. Because the girder spacings had minimal variation, the self-weight of the wearing surface was assumed constant using an average concrete I-girder and steel girder spacing of 2.45 m. Similarly, an average width of 1.2 m was used for concrete box girders. The wearing surface asphalt thickness had little variability as a function of the span length and therefore the slope and R^2 values have no statistical meaning.

Eight out of the ten bridge drawing sets that were received from MI were designed using the HSS-25; however, one steel girder bridge and one precast prestressed concrete box girder bridge were designed with the HSS-30 live load model. This difference may slightly affect the estimate of the self-weight of these two cross sections, but it is not considered to be significant for the calibration of the regression models.

4.4.3.4 Resistance Model

The nominal design resistance (Rn) was calculated according to the AASHTO LRFD design equation (Equation 2) considering the dead load effects established from the regression models and the live load effects from the HSS-25 live load model. The moment resistance was calculated at midspan and the shear resistance was calculated at the support. This resistance is the minimum requirement for the structural capacity of bridge members. Rn is used for the reliability analysis because it assumes that the bridge design exactly satisfies the code requirements, neglecting any overdesign effect.

Figure 6 shows the nominal resistance values in bending and shear that were calculated for the ten representative bridges provided by MI based on the AASHTO LRFD design equation.

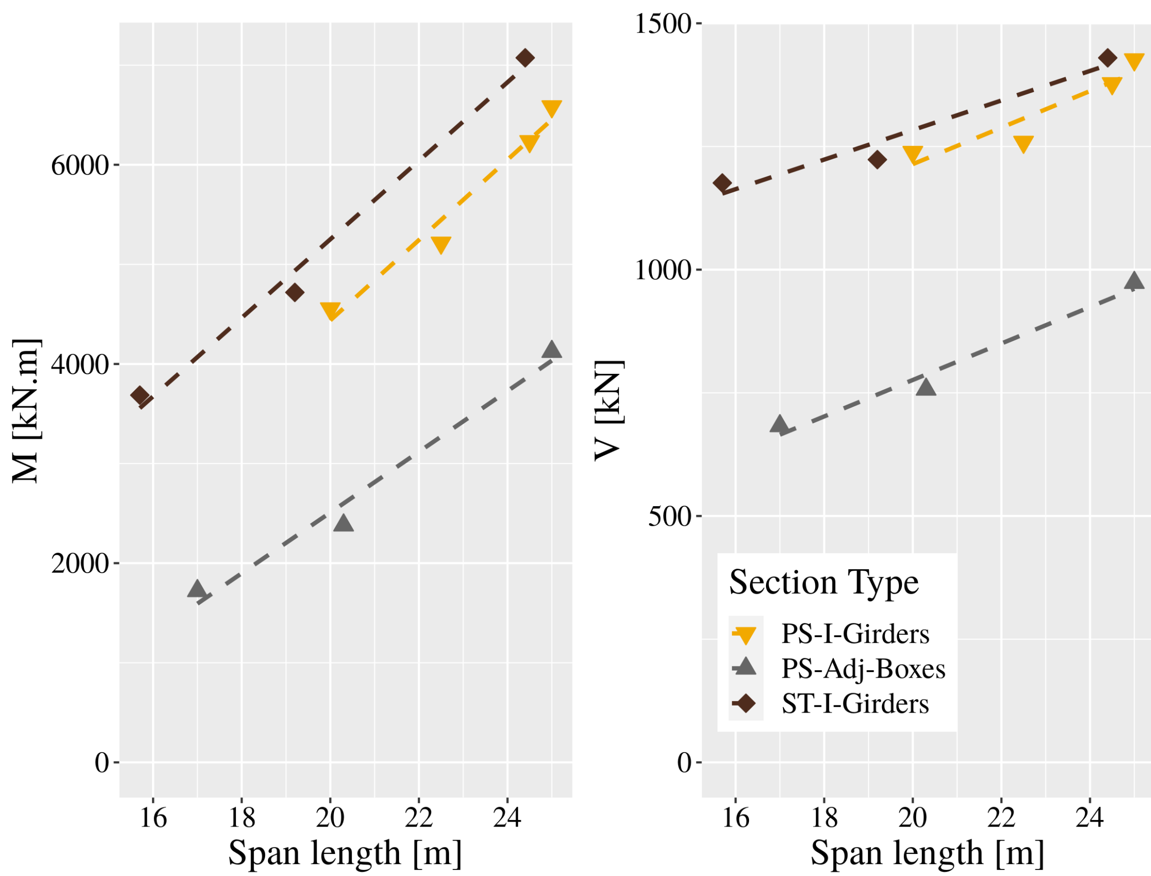


Figure 4-6: Nominal design resistance in bending and shear for ten sample bridges in Manitoba designed according to the HSS-25 load model

4.4.3.5 Reliability Analysis

For the reliability analysis of bridge members, the limit state function Z is expressed as R – S, where R corresponds to the resistance of the member and S is the total effect of the forces acting on it. In the space defined by the variables R and S, the limit state function Z = 0 divides the portion of this space where a structure is considered safe (R >

S) or unsafe ($R < S$). The objective of any reliability analysis is to estimate the probability that $R < S$. Often the probability of failure is expressed through the reliability index, β, which is defined as:

$$\beta = -\Phi^{-1}(P_f)$$

Equation 4-5

where Φ^{-1} is the inverse of the standard normal distribution and P_f is the probability of failure.

Both R and S are random variables described by certain distributions. Typically, for bridge members, R is described by a lognormal distribution with a mean value that is a function of the nominal capacity Rn and a bias factor both in bending and shear (Nowak, 1999). The applied load effect S on a girder is the combined effect of the applied dead loads (DL_1, DL_2 and DW), the live load effect (LL), including the dynamic impact effect, and the distribution factor (DF). Each of these random variables have their own distribution, bias factor and COV. The explicit form of Z reads:

$$Z = R - S = R - [DL_1 + DL_2 + DW + (LL \times DF)]$$

Equation 4-6

Table 4 summarizes the statistical parameters of the random variables described in Equation 6. The nominal value for these random variables was estimated using the procedures and data described in the previous sections of this article for the bridges designed in Manitoba according to the HSS-25 live load model, and from the calibration data of the AASHTO LRFD (Kulicki et al., 2007). As indicated in the last column of Table 4, bias factors and COVs for most of the random variables are obtained from the AASHTO LRFD calibration. Laboratory work using decommissioned bridge girders has been conducted in Manitoba to determine the bias and COV for concrete girder types (Mills, 2009); however, to promote comparison between the Manitoba-based live load statistics and the statistics used to calibrate the AASHTO LRFD code, the AASHTO LRFD specified statistical parameters were considered for the reliability analyses.

Table 4-4: Summary of statistical parameters for the reliability analysis

Random Variable	Nominal Value	Bias Factor	Coefficient of Variation	Distribution/ Reference
Bending Resistance Steel Members, Rn	Equation 2	1.12	0.10	Lognormal (Nowak, 1999)
Shear Resistance Steel Members, Rn	Equation 2	1.14	0.10	Lognormal (Nowak, 1999)
Bending Resistance Prestressed Concrete Members, Rn	Equation 2	1.05	0.07	Lognormal (Nowak, 1999)
Shear Resistance Prestressed Concrete Members, Rn	Equation 2	1.15	0.14	Lognormal (Nowak, 1999)
Factory-made Materials, DL_1	Table 3	1.03	0.08	Normal (Nowak, 1999)
Cast-in-place Materials, DL_2	Table 3	1.05	0.10	Normal (Nowak, 1999)
Wearing Surface, DW	Table 3	1.00	0.25	Normal (Nowak, 1999)
Manitoba Live Load Effect, LL	Projection of Table 2 data	1.10	0.10	Gumbel Type 1
AASHTO LRFD Live Load Effect, LL	HL-93	1.18	0.18	Lognormal (Kulicki et al., 2007)
Distribution Factor, DF	AASHTO LRFD	1.0	-	Deterministic

Two sets of reliability analyses were performed with respect to Equation 6 using the Rackwitz-Fiessler FORM algorithm: one with respect to the probabilistic load model used for the calibration of the AASHTO LRFD specifications and the other with respect to the projected live load distribution computed in Manitoba using Equation 2 (Rackwitz & Fiessler, 1978). The FORM algorithm was written in R (R Core Team, 2018). The reliability analysis was performed under the effect of the live load statistics used for the calibration of AASHTO LRFD to see how far the reliability index of bridges designed in Manitoba using the HSS-25 live load model varied from the target reliability index of $\beta_T = 3.5$ specified in AASHTO LRFD (Nowak, 1999). The Rackwitz-Fiessler FORM algorithm is the same reliability method used for the calibration of the AASHTO LRFD standards and allows for the treatment of nonnormal random variables like those listed in Table 4 (Kulicki et al., 2007).

4.5 RESULTS AND DISCUSSION

This section summarizes and discusses key findings obtained from the reliability analysis of three representative Manitoba bridge types designed according to the HSS-25 live load model.

4.5.1 Results

Reliability analyses were conducted for simply supported precast prestressed concrete box girders, precast prestressed concrete I-girders and steel girders over the span length range of 16 to 26 m. The reliability indices produced in Table 5 were obtained by considering the live load distribution and statistics used for the calibration of the HL-93 live load, whereas Table 6 values consider the projected live load distribution derived from the Manitoba truck surveys on PTH 1. Changes in the reliability index over the range of span lengths are generally not linear because for any given change in the span length, there is also a corresponding change in the design load effect and the self-weight of the members.

Table 4-5: Reliability index values with respect to the AASHTO LRFD live load statistics

Span [m]	Precast Prestressed Concrete Box Girders		Precast Prestressed Concrete I-girders		Steel Girders	
	Bending	**Shear**	**Bending**	**Shear**	**Bending**	**Shear**
16	4.87	4.9	4.85	4.88	4.69	4.99
18	4.91	4.89	4.89	4.89	4.72	5
20	4.94	4.84	4.93	4.87	4.74	4.99
22	4.91	4.77	4.93	4.83	4.71	4.96
24	4.81	4.68	4.87	4.78	4.65	4.91
26	4.74	4.58	4.86	4.73	4.62	4.86
Average	**4.86**	**4.78**	**4.89**	**4.83**	**4.69**	**4.95**
Standard Deviation	**0.08**	**0.13**	**0.03**	**0.06**	**0.05**	**0.06**

Table 4-6: Reliability index values with respect to the Manitoba PTH 1 truck live load statistics

Span [m]	Precast Prestressed Concrete Box Girders		Precast Prestressed Concrete I-girders		Steel Girders	
	Bending	Shear	Bending	Shear	Bending	Shear
16	5.35	5.07	5.33	5.05	5.15	5.16
18	5.32	4.94	5.31	4.94	5.11	5.04
20	5.19	4.73	5.18	4.76	4.99	4.84
22	4.95	4.54	4.95	4.59	4.76	4.66
24	4.76	4.39	4.79	4.47	4.6	4.54
26	4.6	4.23	4.66	4.34	4.47	4.39
Average	**5.03**	**4.65**	**5.04**	**4.69**	**4.85**	**4.77**
Standard Deviation	**0.31**	**0.32**	**0.28**	**0.27**	**0.28**	**0.30**

Figure 7 depicts the difference between the reliability indices produced with respect to the projected Manitoba truck data statistics (Table 6) and those produced with respect to the AASHTO LRFD live load statistics (Table 5) for each structure type and live load effect as a function of span length.

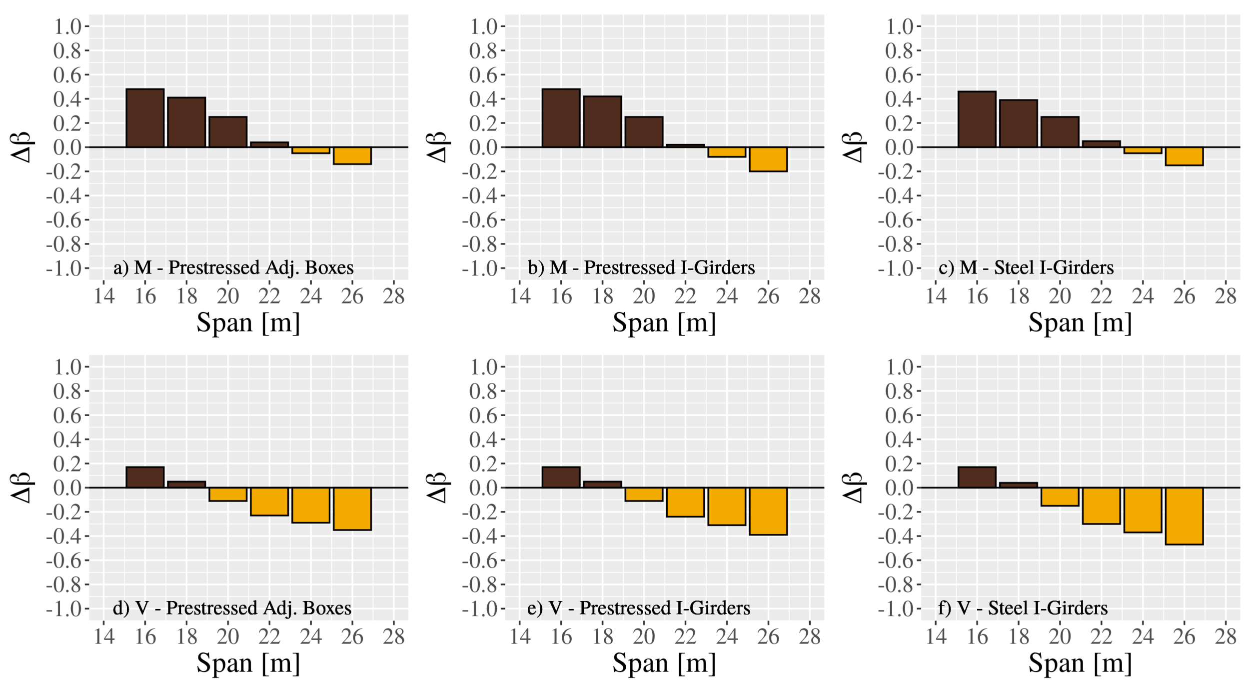


Figure 4-7: Difference between the reliability indices resulting from the Manitoba live load statistics and the AASHTO LRFD live load statistics

4.5.2 Discussion

The results in Tables 5 and 6 and Figure 7 reveal the following:

- By conducting a reliability analysis considering the live load statistics that were used for the calibration of the AASHTO LRFD code, a comparison can be made about the reliability index of bridges designed according to the HSS-25 live load model in Manitoba to the structures referenced during the American code calibration. For the structures and span ranges analyzed, the reliability index values of the Manitoba designed structures exceed the 75-year lifetime target reliability index of 3.5 indicated by AASHTO LRFD (Kulicki et al., 2007) and the average reliability indices range from 4.69 to 4.95 (Table 5). The average reliability index values exceed the target reliability index, which also represents an average value.
- Similarly, the reliability analysis results in Table 6 show that representative bridges designed according to AASHTO LRFD and the HSS-25 live load model under the effect of Manitoba live load statistics exhibit a higher safety margin than the code specified target reliability index over the considered span lengths. The average reliability indices range from 4.65 to 5.04 (Table 6). These results therefore indicate that the previously unanalyzed HSS-25 live load model used to design bridges according to AASHTO LRFD has a level of safety that exceeds the code requirement for the girder types considered. Overdesigning the bridge member cross sections using 25% additional load with respect to the HS-20 axle loads only increases the overall material cost of the girders by about 5%, which for typical highway bridges is considerably smaller than the overall bridge cost (Fiorillo & Ghosn, 2019). This is a negligible material cost increment compared to the possible longer-term benefits received in terms of additional safety provided to the members; however, the total economic impact of this conservatism on a network-wide scale may require further assessment. Regardless, the studied structure types may have the capacity to carry ultimate live load effects greater than those currently imposed by the projected Manitoba truck fleet.

- A general reduction in the safety margin was observed with increasing span length for both sets of reliability indices. This may suggest a correlation between the dead to live load ratio and the reliability index value. Similar patterns were observed during the AASHTO LRFD calibration, which also noted that increasing dead to live load ratios resulted in reduced reliability values (Kulicki et al., 2007).
- The reliability values in Tables 5 and 6 indicate that there is typically an elevated reliability index when comparing the average span bending load effects to the shear load effects. The structure type and the live load effect (moment vs shear) influences the size of the cross section and resulting self-weight, and the selection of the resistance factor; the effect of this is shown in the general design equation (Equation 2).
- The standard deviation of the reliability indices for the analyzed bridges computed using the AASHTO LRFD live load statistics are noticeably smaller than those computed using the Manitoba live load statistics. On average, the standard deviation of the reliability indices for the AASHTO LRFD statistics is approximately 88% smaller. This occurs because the safety factors of the AASHTO LRFD ultimate limit state were calibrated to provide uniform reliability indices across different span lengths under the effects of the HL-93 live load model. In fact, the purpose of a reliability-based code calibration is to determine a set of load and resistance factors such that the resulting reliability indices are uniform regardless of bridge type and span length. The recalibration of specific load and resistance factors for the HSS-25 live load model would lead to more consistent target reliability indices with varying span length.
- The average reliability indices calculated when considering the Manitoba live load effects (Table 6) are similar to those calculated when considering the AASHTO LRFD live load effects (Table 5). However, the Manitoba-based indices are larger for shorter spans and become smaller than the AASHTO LRFD-based indices as span length increases, as shown in Figure 7. Because the resistance side of the equation remains unchanged between the two sets of indices, this result shows the effect of differences in the live load bias factor and distribution,

as well as variations in the nominal live load effect with increasing span length. The projected Manitoba nominal live load produces more severe load effects than the HL-93 for longer spans, resulting in a reduced reliability value.

- In general, the change in reliability indices—between the considered live load models and with varying span length—illustrates the importance of monitoring how traffic live loads and bridge design code live load models evolve over time and understanding how these changes impact the level of safety of bridge infrastructure.

In addition to the analytical findings from the reliability analysis, a major contribution of this article is the application of unique, longitudinal truck weight and axle spacing datasets collected over a two-decade period in Manitoba that were used to develop the projected maximum live load effects. By using this data, a more representative assessment of the reliability of local bridge types subjected to observed truck loads on the Trans-Canada Highway resulted. Limitations in the availability of reliable truck data have posed challenges during code generation in the past, as was the case during the development of the AASHTO LRFD code, which based its statistical parameters on a 1975 Ontario truck survey due to the lack of United States data at the time of calibration (Kulicki et al., 2007). Increasing truck volumes and differing configurations in the United States have resulted in questions arising about the representativeness of the previous Ontario data, but despite improved American truck weight data quality and quantity in recent years, this information has not been used to update the bridge design loads (Sivakumar et al., 2011).

4.6 LIMITATIONS

The findings presented in the previous section should be interpreted in the context of several limitations. This section describes these limitations and examines their impact on the findings.

4.6.1 Case Study Approach

A case study approach was used to select three representative bridge types and a corresponding span length range that was considered for the reliability analysis.

Although this approach does not encompass the entire range of bridge types and spans in Manitoba, and therefore limits the transferability of the results, these selections were considered reasonably representative of structures on major trucking routes in the province and coincided with the truck weight data collection sites on PTH 1. Further study into additional structure types and span lengths following the presented methodology would expand upon the findings herein.

4.6.2 Truck Data Source Limitations

The locally collected truck data was also subjected to several unavoidable data-related limitations. First, because axle spacing is not typically measured at static weigh scales, the axle spacing was determined from WIM stations located on PTH 1. Previous studies have found WIM devices to be accurate for determining inter-axle spacings and distinguishing between axle group types; therefore, the WIM measurements are considered representative of the desired accuracy, especially given that the axle group spacings used in the analysis were rounded to the nearest 100 mm (Regehr et al., 2009). The consistency in the WIM results over the studied time period was demonstrated by comparing data collected from two locations in 2006 and 2017, which showed maximum differences of 5.5% and 4.0% for average axle spacings within a group and average interaxle spacings, respectively. The average value of the axle spacing data was used for the analysis, as listed in Table 1, and it is acknowledged that there is a distribution of possible spacings for the various truck configurations. The use of the average value was considered appropriate for the initial reliability analysis; however, future sensitivity studies could randomly assign values from the spacing distributions to assess the impact of this assumption on the reliability indices.

Second, uncertainties in the analysis may have resulted from the fact that data was collected with short-term surveys. Although more than twenty thousand trucks were considered over two decades, each year only contained a limited number of sampled trucks compared to the annual population. Low sample sizes may have influenced the 2016 data results shown in Figure 5, as this year had the fewest number of days of data collection and therefore may have been impacted by seasonal hauling patterns. Despite this, the local and timely nature of the data allowed for development of a live load model for Manitoba, which is considered more representative than other truck weight and spacing datasets historically collected in neighbouring provinces, such as the 1975 Ontario database.

Finally, limitations surrounding the representativeness of the results due to the enforcement intensity at the time of collection could not be quantified. Previous research has stated that high levels of enforcement lead to near complete compliance (Clayton et al., 1998; Fiorillo & Ghosn, 2016). It is also reasonable to assume that overloaded trucks may have taken measures to purposefully avoid permanent weight scale stations and the data may have underrepresented the actual weights of trucks crossing bridge infrastructure. However, this impact was considered minor and an analysis of enforcement and/or avoidance levels was outside the scope of this article. It is also recognized that the analyzed weigh scale data did not generally include permit vehicles and these vehicles may also cause more severe load effects on bridge infrastructure. Limited information on the weight and operation of permit vehicles in Manitoba prevented assessment of these effects and the scope was instead focused on typical truck movements on PTH 1 over the past two decades. Future analysis could consider the effects of permit vehicles on the reliability of Manitoba bridges (considering both the HSS-25 and the HSS-30 live load models).

4.6.3 Reliability Analysis Approach

The nominal resistances used in the reliability analysis were established from the dead load regression models based on representative bridge drawing sets provided by MI. A possible limitation is related to further application of the regression models outside the calibration limits of the span lengths. To reduce this effect, a larger number of bridge drawings could be used to improve the quality of the models; however, when variation in the cross section was present, the R^2 value was high for the dead load parameters over the examined span range and therefore the limited ten drawing sample is not expected to affect the reported results. Additional drawing sets could be valuable for considering other structure types and span ranges, including continuous spans, in order to assess negative moment regions.

4.7 CONCLUSIONS

For the last several decades, bridge structures in Manitoba have been designed in accordance with AASHTO LRFD and the Modified HSS-25 live load model. Although bridge performance has historically been satisfactory, a technical assessment of the reliability of structures has not been performed to date. This article presented a case study approach to selecting typical bridge types and developing Manitoba-specific live load statistics based on observed truck data, followed by reliability analyses to determine the level of safety of the selected bridge types designed according to the HSS-25 live load model. The analysis considered the live load statistics used for the calibration of the AASHTO LRFD code and the locally established live load statistics to determine the effects on structure reliability.

The analysis revealed that simply supported precast prestressed concrete box girders, precast prestressed concrete I-girders and steel girders designed according to AASHTO LRFD and the HSS-25 live load model over the span range of 16 to 26 m consistently maintained a reliability index that exceeded the target reliability index of β = 3.5, as specified in the design code. The average reliability index values resulting from the AASHTO LRFD live load statistics ranged from 4.69 to 4.95 and the average reliability index values resulting from Manitoba truck live load statistics ranged from 4.65 to 5.04. A general reduction in the safety margin was observed with increasing span length. This reduction was likely influenced by the increasing dead to live load ratio.

For all three bridge types, the standard deviation of the reliability indices was lower for the AASHTO LRFD live load statistics, demonstrating the effect of calibrated load and resistance factors for the HL-93 model. Although the range of reliability values were comparable for the different live load statistics, the Manitoba-based statistics typically exhibited a lower probability of failure, especially for shorter spans, indicating the potential capacity of the analyzed structure types and spans to support larger truck live load effects.

Despite some unavoidable, data-related limitations, the overall analysis was facilitated by a unique, longitudinal truck weight and spacing database that consisted of more than

20,000 trucks measured over a two-decade period in Manitoba. This database provided a reliable indication of the typical truck traffic experienced by bridges along the Trans-Canada Highway. The findings confirmed the safety of the examined structure types designed with the uncalibrated HSS-25 live load model and provide an opportunity for future analysis to extend the results to other bridge types and span lengths used in Manitoba.

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5.0 **Conclusions**

This chapter summarizes the key findings and implications of this research and also recommends opportunities for future research.

5.1 SUMMARY OF KEY FINDINGS AND IMPLICATIONS

Understanding the complexities and impacts of truck size and weight regulations has been a topic of study in North America and globally, driven by desires to improve the uniformity and productivity of road freight transportation (Di Cristoforo et al., 2012; Knight et al., 2008; Ortega et al., 2014). However, methods to reliably estimate the effects of regulatory change on infrastructure, safety, emissions and the environment, and other freight modes are difficult to establish. Despite this challenge, Canadian agencies decided to implement several national scale TSW regulatory changes over the past five decades, including the 1974 WCHSP, the 1988 RTAC MoU and special permitting of LCVs, and have realized certain economic benefits from this decision (Woodrooffe et al., 2010). By analyzing data collected in Canada following these changes, observations about the direct impacts resulting from TSW limit changes can be made to clarify concepts and provide transferrable insights to other jurisdictions.

This research presents evidence of the impacts of TSW regulation changes on bridge live load models, truck operating gross vehicle weights and bridge structure reliability, specifically focusing on the context of Manitoba, Canada. In particular, this thesis looked to address the following three primary research questions:

- How have bridge live load models evolved in concert with truck weight limits?
- How have truck gross vehicle weights evolved as a function of truck weight limits?
- How has bridge reliability been impacted by changes in live load models and truck operating weights in Manitoba?

5.1.1 Impacts on Bridge Live Load Models

Over the past five decades, TSW limits have undergone modifications, typically increases, which have affected the operating truck fleet that in-turn loads bridge infrastructure. During this same period, bridge design codes in North America and the specified live load model have also evolved, especially following the emergence of load and resistance factor design methods. Although the relationship between legal truck weight limits and bridge code live load models appears self-evident, on occasion there has been limited collaboration between truck size and weight policy makers and bridge code authors.

The historical review presented how bridge live load models have evolved in concert with truck weight limits. It revealed instances in which bridge codes responsively accounted for expected truck weight policy changes, which happened during the development of the 1988 CSA S6 design code, as well as situations where code modification lagged regulatory change by several years, as was the case for the 1979 OHBDC. Despite this, eventual evaluations have not necessitated major code revision, which speaks to the potential conservatism within bridge design methods. However, as the Canadian truck fleet continues to diversify, especially with increased LCV implementation, future assessment of the applicability of live load models may be required.

The research concluded that a collaborative relationship between parties supporting additional truck weight increases and bridge design code authors would be proactive and important for continuing to ensure a uniform level of safety for Canadian bridge infrastructure.

5.1.2 Impacts on Truck Gross Vehicle Weights

In order to predict the effect of TSW regulations on bridges (amongst other applications), an understanding of the changes to the truck operating fleet is required. Although some methods have been presented to predictively model these effects, a retrospective study over a five decade period provides a unique opportunity to comment directly on the observed impacts, particularly as they relate to understanding how truck operating gross vehicle weights will correspondingly change when weight limits are relaxed, how those changes may differ for various truck configurations and how long it may take for the trucking industry to adopt the changes. Key findings from the analysis are as follows:

- As a result of three purposeful truck regulatory changes in Canada, trucks are now operating at heavier weights today than they did five decades ago and carriers have in fact taken advantage of increased payload opportunities and new available truck configurations. Similarly, periods with minimal regulatory change resulted in a relative stability in truck operating weights.
- The truck configuration and the type of commodity hauled resulted in markedly different GVW distribution changes following allowable weight limit increases. B-doubles, which commonly haul weigh-out commodities were found to take advantage of weight limit increases, whereas Turnpike doubles, which typically haul cube-out commodities, were less sensitive to such increases. This demonstrated that the trucking industry will leverage the productivity benefits afforded by a diverse fleet mix.
- The industry's response time to increased allowable weight limits appeared to be influenced by the ease of implementation. While immediate increases in GVW were observed for weigh-out trucks responding to more relaxed weight limits for a given truck configuration, additional time was required when new, more productive truck configurations were introduced. A delay of approximately three to four years was observed for carriers to adjust their fleets and logistics practices to take advantage of increased limits with new configurations.
- Exogeneous trends can also impact the truck GVW distributions. The reduction in average freight density and reduction of truck tare weight appeared influential for all truck configurations.

Despite data related limitations and the geographic scope of the findings, the evidence presented contributes new insights and addresses gaps and uncertainties that are commonly applicable to prediction models. Direct examples of freight carrier responses to TSW regulation changes as it relates to the type of commodity hauled and the available fleet mix can be informative to other jurisdictions desiring to increase TSW limits. Further, increased understanding surrounding the expected changes in truck operating weights is especially valuable for predicting the impact on bridge structure reliability.

5.1.3 Impacts on Bridge Structure Reliability

Truck operating weights change over time, often as a result of TSW regulation modifications or due to a desire by individual provinces to allow heavier vehicles to operate on their highway network, typically in response to industry specific live load requirements. To account for this, several provinces in Canada have specified their own design live load model, modifying the one provided in the national bridge design code, to ensure that the maximum probability of failure of existing and new bridge infrastructure is limited to within acceptable thresholds. A case study approach to determine the reliability of three typical bridge types in Manitoba designed in accordance with a local live load model specified by Manitoba Infrastructure, the HSS-25, and subject to both observed historical truck live loads and also to the live load effects used to calibrate the AASHTO LRFD code, produced the following conclusions:

- Simply supported precast prestressed concrete box girders, precast prestressed concrete I-girders and steel girders designed according to AASHTO LRFD and the HSS-25 live load model over the span range of 16 to 26 m consistently maintained a reliability index that exceeded the target reliability index of $\beta = 3.5$, as specified in the design code.
- A general reduction in the safety margin was observed with increasing span length. This reduction was likely influenced by the increasing dead to live load ratio.
- Although the range of reliability values were comparable for the different live load statistics, the Manitoba-based statistics typically exhibited a lower probability of failure, especially for shorter spans, indicating the potential capacity of the analyzed structure types and spans to support larger truck live load effects.
- The overall analysis was facilitated by a unique, longitudinal truck weight and spacing database that consisted of more than 20,000 trucks measured over a

two-decade period in Manitoba. Limitations in the availability of reliable truck data have posed challenges during bridge code generation in the past. This database provided a representative and timely indication of the typical truck traffic experienced by bridges along the Trans-Canada Highway.

In general, the change in reliability indices—between the considered live load models and with varying span length—illustrates the importance of monitoring how traffic live loads and bridge design code live load models evolve over time and understanding how these changes impact the level of safety of bridge infrastructure.

5.2 RECOMMENDATIONS FOR FUTURE RESEARCH

Although the research addressed several aspects related to the impacts of TSW regulations, additional extensions of the work may be valuable.

- Chapter 3 provided observations and insights on how TSW limit modifications effect truck operating gross vehicle weight distributions. Although the results provide evidence of the impacts, further research is required to compare the theoretical output from a typical freight demand model and modal diversion analysis by truck configuration to the observations presented in the study. This exercise would allow the observed impacts to serve as validation for the predicted model and may provide transferrable insights for future model development and evaluation in general.
- Enforcing TSW limits is important for protecting infrastructure, reducing wear on highway pavements and limiting bridge failures due to illegally overloaded vehicles. The presented research references static truck weigh scale data and acknowledges that illegally operating trucks may have avoided permanent scales and remained undetected in the dataset due to unknown levels of enforcement at the time of data collection. Future research on how enforcement levels impact truck operating weights would provide added insight towards understanding the operating patterns regardless of the specified truck size and weight limits. The relationship between enforcement levels and the probability and extent of illegal

operations would also be valuable for establishing live load statistics for bridge reliability analyses.

- Although several provinces have introduced their own live load models in recent decades, the bridge design code specified live load models have remained unchanged since 1994 and 2000 for AASHTO and CHBDC, likely as a result of minimal changes in TSW limits in both the United States and Canada since that time. However, as permit vehicle use and LCV operations become more widespread across Canada, additional unique truck configurations may require structural assessment to determine if the current live load models and bridge design codes adequately address the loading conditions, whether for ultimate limit states, fatigue or otherwise. Additional truck weight surveys, design checks and reliability analyses are recommended to ensure adequate code calibration and reliability.
- The reliability analyses provided in Chapter 4 were conducted for three typical bridge types in Manitoba for simply supported structures over a defined span length range. Additional research opportunities include an extension of the method to consider all of the structure types in the Manitoba Infrastructure bridge inventory, over an increased span length range for different support conditions, including multi-span continuous structures. This research would allow additional conclusions to be made about the performance of the HSS-25 live load model considering the current live load effects in previously unanalyzed situations, such as in negative moment regions.
- Finally, it is recognized that monitoring the evolution of traffic loads over time and leveraging up to date truck size and weight databases would be valuable for understanding the current loading that existing bridge infrastructure must safely withstand. Technologies, such as bridge weigh-in-motion (BWIM) devices, may prove valuable in these efforts. The reliability analyses presented in Chapter 4 was used to determine the level of safety of new bridges designed according to the AASHTO LRFD code with an HSS-25 live load model by assuming that the bridge nominal resistance exactly satisfied the requirements of the factored

loads. However, future research could also be conducted to evaluate an existing bridge for its reliability index for specific limit states by substituting the actual bridge resistance determined from a load test into the same type of reliability analysis. These results may be beneficial for assessing the level of safety resulting from the evaluation trucks that are specified in the bridge design code and also the methods used to conduct the load test.

Overall, the focus of this work is to highlight the influential effect of evolving truck traffic loads on infrastructure and promote increased collaboration to bridge the gap between transportation and structural engineers. The proposed research will help to continue these efforts.

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7.0 Appendix A: Reliability Index Calculation Procedure

The reliability analysis procedure presented in Chapter 4 used a program written in R to calculate the reliability index values. The following is a sample calculation for a steel girder bridge with a simply supported span length of 24 m. Additional detail and references for this process are included in Chapter 4.

7.1 LIVE LOAD MODEL

7.1.1 Conduct Influence Line Analysis for Observed Manitoba Truck Data

The Manitoba truck weight database was established from truck weigh scale data and WIM spacing data. For each truck in the database, an influence line analysis was completed to determine the maximum moment and shear effects over the specified span length range from 16 – 26 m.

The following truck configuration produced the maximum moment and shear effects for the sample calculation span length of 24 m. The truck is an 8-axle B Double and has weights exceeding the specified limits in the MoU (5% on the drive tandem axle and 20% on the tridem axle group). However, these values did not exceed the 50% overloaded erroneous value filter and were therefore considered valid axle group loads.

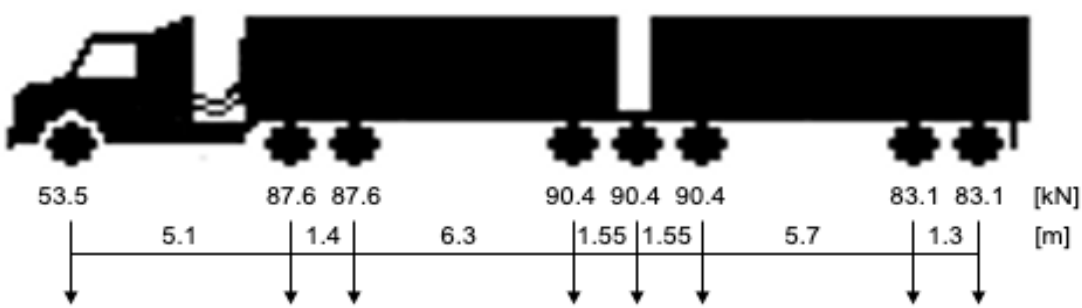

Figure 7-1: Observed truck producing the maximum load effects for a span length of 24 m

The maximum applied live load effects from the truck were determined to be 2130.45 kN.m for positive moment and 390.90 kN for shear for the simply supported span length of 24 m.

The maximum daily moment and shear load effects considering all of the trucks in the database and a span length of 24 m are shown in Figure 2.

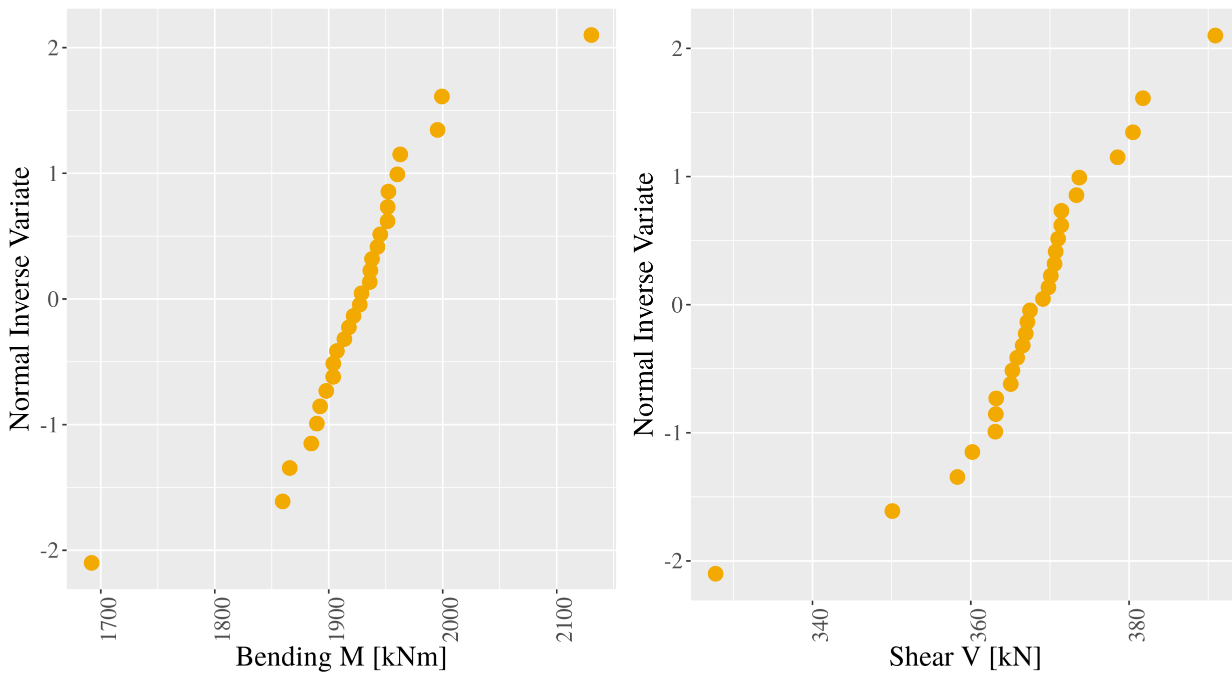

Figure 7-2: Maximum daily moment and shear load effects resulting from Manitoba trucks for a span length of 24 m

The truck in Figure 1 produced the most severe loading events on the Figure 2 plots.

7.1.2 Obtain Statistical Parameters for Daily Maximum Live Load Effects

After the daily maximum live load effects were determined from the truck database, a frequency analysis was performed to fit the upper tail of the distribution to a normal distribution. L-Moment estimators were used to determine the mean and variance of the distribution of the daily maximum load effects (Stedinger et al., 1993).

Unbiased probability-weighted moments (PWMs) are estimated as follows, where X is the ordered observations of moment or shear daily maximum load effects.

$$b_0 = \bar{X}$$

Equation 7-1

$$b_1 = \sum_{j=1}^{n-1} \frac{(n-j)X_j}{n(n-1)}$$

Equation 7-2

L-moment estimates for a Normal Distribution can be calculated in terms of PWMs with the following equations (Stedinger et al., 1993):

$$mean\ (\mu) = \lambda_1 = b_0$$

Equation 7-3

$$standard\ deviation\ (\sigma) = \lambda_2 \times \sqrt{\pi} = (2b_1 - b_0) \times \sqrt{\pi}$$

Equation 7-4

Table 1 presents the statistical parameters that were determined from the daily maximum load effect data and Equations 1 to 4.

Table 7-1: Daily maximum bending and shear L-Moment estimates and statistical parameters

Parameter	Moment	Shear
$\mathbf{b_0}$	1933.9	369.8
$\mathbf{b_1}$	980.0	186.8
μ	1933.9 kN.m*	369.8 kN*
σ	46.2 kN.m	6.8 kN
COV	0.024*	0.018*
$\mathbf{R^2}$	0.894*	0.953*

Note: * indicates values specified in Table 4-2 for Span = 24 m

As shown by the linear nature of the line of points on normal probability paper for bending and shear (refer to Figure 2), and the high R^2 value in Table 1, a normal distribution well describes the upper tail portion of the daily maxima bridge response.

7.1.3 Determine Two-Lane Loading Effects

Simultaneous occurrence of two trucks moving in the same direction can be approximated using a convolution integral, expressed as follows:

$$f_i\,(x) = \int_{-\infty}^{\infty} f_2\,(x - z) \cdot f_1(z)\,dz$$

Equation 7-5

where $f_i(x)$ is the probability density function of the convolution for the response x, while $f_1(\ldots)$ and $f_2(\ldots)$ are the probability density functions of the daily maxima effect on lanes 1 and 2, respectively. This integral can be solved using software to determine the two-lane loading effect. The graphical output of this is shown in Figure 3, assuming a minimum COV of 0.15. Note that the single lane loading distribution represents the normal distribution that was established from the parameters in Table 1.

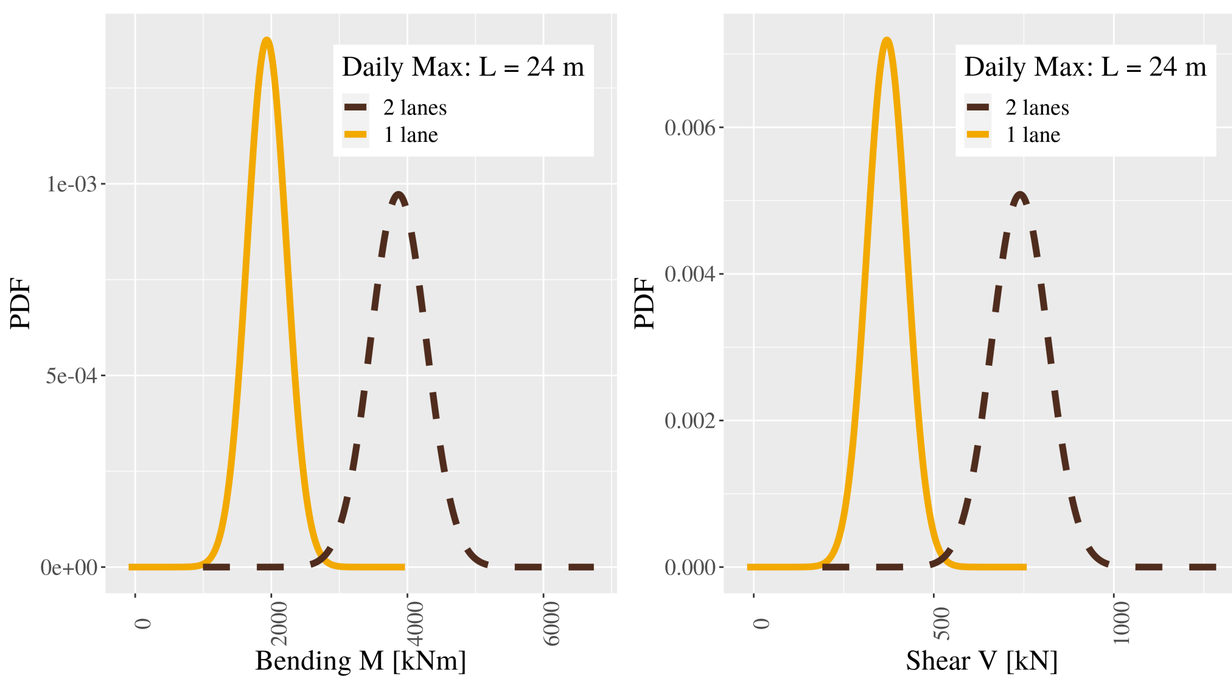

Figure 7-3: Probability density functions for single lane and two lane daily maximum load effects resulting from Manitoba trucks for a span length of 24 m

7.1.4 Statistically Project the Daily Live Load Effects Using Extreme Value Theory

A statistical projection of the observed daily maximum load effects using extreme value theory was conducted to estimate the daily maximum load effect over the design life of the structure.

Assuming that the initial variate of the daily maximum load effect is time invariant, the cumulative distribution at time t, $F_t(x)$, can be obtained using the following equation:

$$F_t(x) = F_i(x)^{Nev}$$

Equation 7-6

where N_{ev} = total number of events = 365 days x 50 years = 18,250. Explanation for the selection of a 50-year time period for this study is provided in Chapter 4.

By applying a power of N_{ev} to each value in the cumulative distribution, the CDF is compressed towards the upper maximum value, as shown in Figure 4.

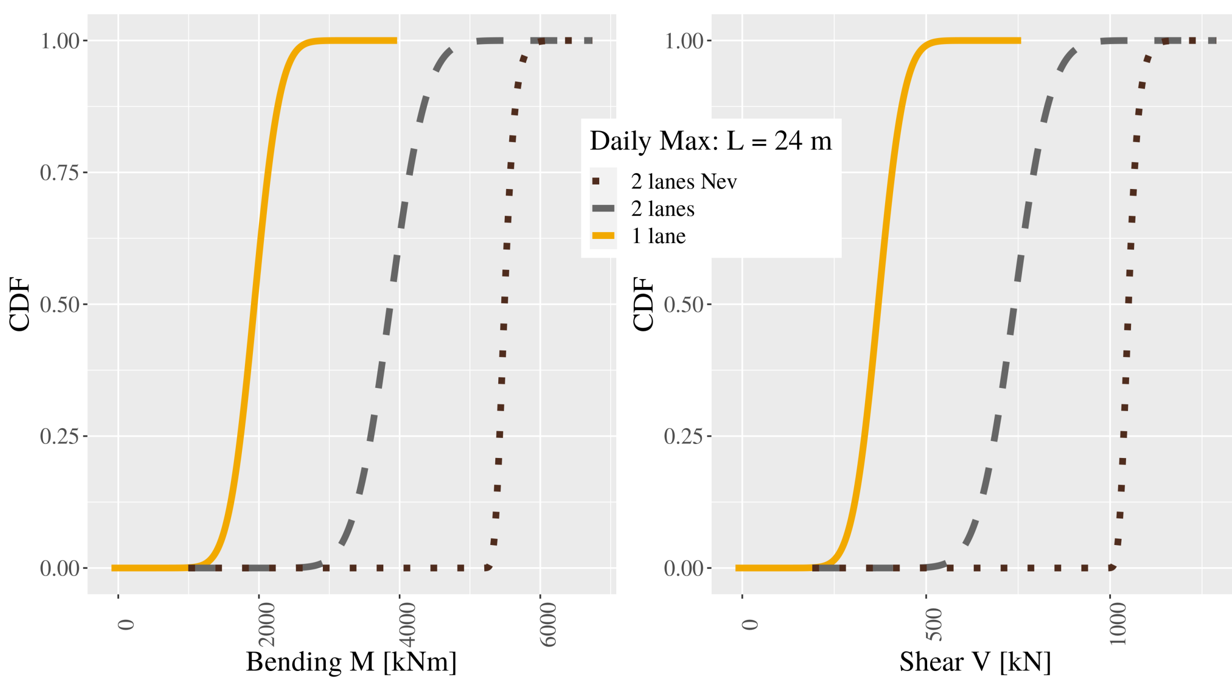

Figure 7-4: Cumulative distribution functions for single lane, two lane and the two lane projection (for Nev = 18,250) of daily maximum load effects resulting from Manitoba trucks for a span length of 24 m

According to extreme value theory, if a distribution of the initial variable has a general normal distribution with mean, μ, and standard deviation, σ, then the maximum value after N_{ev} repetitions asymptotically approaches a Gumbel Type 1 extreme value distribution (Ang & Tang, 1984). The mean maximum daily load effects shown in Figure 4 are listed in Table 2 (CDF = 0.5).

Table 7-2: Mean daily maximum load effects for two lanes loaded and a span length of 24 m resulting from Manitoba trucks

	Moment (kN.m)	COV Moment	Shear (kN)	COV Shear
Initial value (t = 0)	3867.9	0.11	739.6	0.11
Gumbel Type 1 Projected Value (t = 50 years)	5508.0	0.022	1053.2	0.022

To account for the effects of impact on the live load, a bias factor of 1.1 and COV of 0.10 was used, as is specified in the AASHTO LRFD calibration (Kulicki et al., 2007). A final COV of 0.10 for the live load (including impact) was calculated as follows:

$$COV_{LL+IM} = \sqrt{{COV_{LMax}}^2 + {COV_{IM}}^2} = \sqrt{0.022^2 + 0.10^2} = 0.10$$

Equation 7-7

7.1.5 Determine Distribution Factors Based on AASHTO LRFD

Distribution factors (DF) are used in bridge codes to distribute the live load effects to the girders. According to AASHTO LRFD, the DF for interior steel girders can be calculated using the following simplified equations for moment and shear:

$$One\ Design\ Lane\ Loaded, DF_1 = 0.06 + \left(\frac{S}{14}\right)^{0.4} \left(\frac{S}{L}\right)^{0.3} \left(\frac{K_g}{12L{t_s}^3}\right)^{0.1}$$

$$Two\ Design\ Lanes\ Loaded,\ DF_2 = 0.075 + \left(\frac{S}{9.5}\right)^{0.6} \left(\frac{S}{L}\right)^{0.2} \left(\frac{K_g}{12L{t_s}^3}\right)^{0.1}$$

Equation 7-8: Live Load Distribution Factor for Moment in Interior Beams (Table 4.6.2.2.2b-1) (AASHTO, 2017)

$$One\ Design\ Lane\ Loaded, DF_1 = 0.36 + \left(\frac{S}{25}\right)$$

$$Two\ Design\ Lanes\ Loaded,\ DF_2 = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^{2.0}$$

Equation 7-9: Live Load Distribution Factor for Shear in Interior Beams (Table 4.6.2.2.3a-1) (AASHTO, 2017).

Based on the cross sections for the available steel bridge drawings from MI, the following parameters are applicable:

S = average girder spacing from drawings (ft) = 2.45 m / 0.3048 (ft/m) = 8 ft

L = span length (ft) = 24 m / 0.3048 (ft/m) = 78.7 ft

Number of girders = 5

Simplified value for $\left(\frac{K_g}{12Lt_s{}^3}\right)^{0.1} = 1.02$ (for steel girder bridges according to AASHTO LRFD, Table 4.6.2.2.1-3).

The resulting DF are listed in Table 3.

Table 7-3: AASHTO LRFD live load distribution factors for interior steel girders with a span length of 24 m and girder spacing of 2.45 m

	Shear	Moment
One Design Lane	0.68	0.47
Two Design Lanes	0.82	0.66

Two design lanes were considered with the convoluted load effects. In AASHTO LRFD, the multiple presence factor for two loaded lanes is 1.0 (Table 3.6.1.1.2-1) (AASHTO, 2017). AASHTO DF equations should be applied to the effect of a single lane – because convolution accounts for two lanes of loading, the load effect was divided by 2.

Based on the projected two-lane load Extreme Gumbel Type 1 distribution (Table 2), the calculated distribution factors (Table 3), the multiple presence factor and the bias factor for impact, the following mean moment and shear effects resulting from the Manitoba truck data for a steel girder with a span length of 24 m can be calculated as follows:

$$Mean\ Moment$$
$$= Mean\ of\ projected\ extreme\ Gumbel\ Type\ 1\ distribution$$
$$\times\ impact\ bias\ factor\ \times\ DF_m \times 0.5 = 5508\ \times\ 1.1\ \times\ 0.66\ \times\ 0.5$$
$$= 1999\ kN.m$$

$$Mean\ Shear$$
$$= Mean\ of\ projected\ extreme\ Gumbel\ Type\ 1\ distribution$$
$$\times\ impact\ bias\ factor\ \times\ DF_v \times 0.5 = 1053\ \times\ 1.1\ \times\ 0.82\ \times\ 0.5$$
$$= \quad 475\ kN$$

Equation 7-10: Mean moment and shear live load effects for projected Manitoba truck data

Table 7-4: Projected Manitoba truck two-lane live load effects for a steel girder with a span length of 24 m

	Mean Moment (kN.m)	Mean Shear (kN)	COV
Manitoba Live Load Effect	1999	475	0.10

The mean moment and shear for the Manitoba live load statistics calculated in Equation 10 and listed in Table 4 were used for the reliability calculations outlined in Section 7.4.

7.1.6 Live Load Model Considered in AASHTO LRFD Calibration

The live load statistical model for two-lane loading used for the development of AASHTO LRFD consisted of a lognormal distribution with a mean value expressed by the product of the nominal HL-93 live load response and a bias factor. The nominal and mean moment and shear effects for a span length of 24 m are listed in Table 5, along with the statistical parameters provided in the AASHTO LRFD calibration report (Kulicki et al., 2007).

The mean values required for the reliability analysis are calculated as follows:

$$Mean\ Moment = \ Nominal\ HL - 93\ Moment\ \times\ bias\ factor\ \times\ DF_m$$
$$= 2230.9\ \times\ 1.18\ \times\ 0.66\ = 1737\ kN.m$$

$$Mean\ Shear = \ Nominal\ HL - 93\ Shear\ \times\ bias\ factor\ \times\ DF_v$$
$$= 396.7\ \times\ 1.18\ \times\ 0.82\ \times\ 0.5 = \quad 384\ kN$$

Equation 7-11: Mean moment and shear live load effects for AASHTO LRFD live load model

Table 7-5: AASHTO LRFD live load effects and statistical parameters for a steel girder with a span length of 24 m

	Nominal Moment (kN.m)	Nominal Shear (kN)	Bias Factor	Mean Moment (kN.m)	Mean Shear (kN)	COV
AASHTO LRFD Live Load Effect (HL-93)	2230.9	396.7	1.18	1737	384	0.18

7.2 DEAD LOAD MODEL

The dead load parameters were determined from the cross sections of sample bridge drawings designed to the HSS-25 live load model (provided by Manitoba Infrastructure). The following section shows how the regression parameters for dead loads were developed. There were three steel bridge drawings available; the continuous structure used for the following sample calculation had an intermediate span length of 22.5 m with one fixed and one expansion bearing. The distance between inflection points (positive bending effective length) was approximated with a theoretical K value = 0.7. The effective span length used for subsequent calculations was therefore assumed to be 15.7 m.

The self-weight, superimposed dead loads and wearing surface parameters were calculated based on the bridge drawing cross section details, shown in Figure 5.

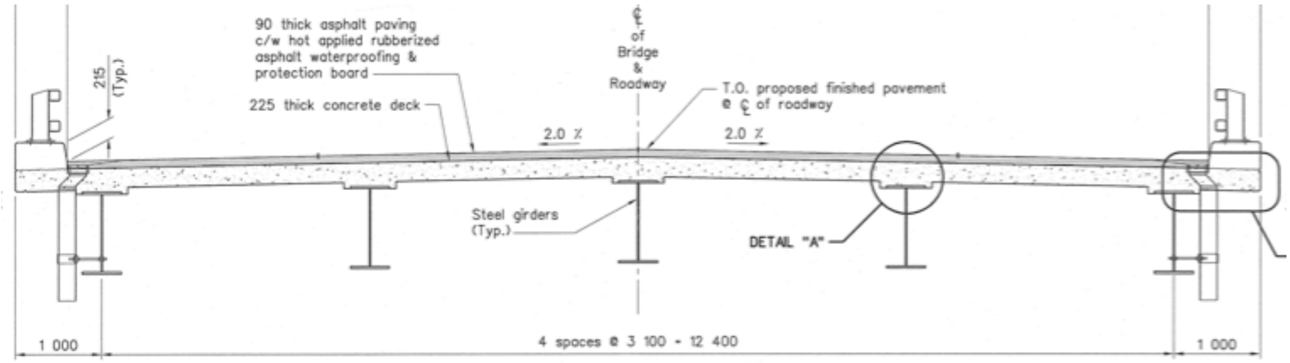

Figure 7-5: Cross section of sample steel bridge provided by Manitoba Infrastructure

7.2.1 Factory-Made Materials, DL_1

The weight of factory-made materials was calculated from the steel girder cross section shown in Figure 6.

$$DL_1 = \rho_{steel} \times XS\ Area = 78.5\ \frac{kN}{m^3} \times \left[\frac{14mm \times 900mm + (25mm + 35mm) \times 450mm}{10^6}\right]$$
$$= 3.1 \frac{kN}{m}$$

Equation 7-12

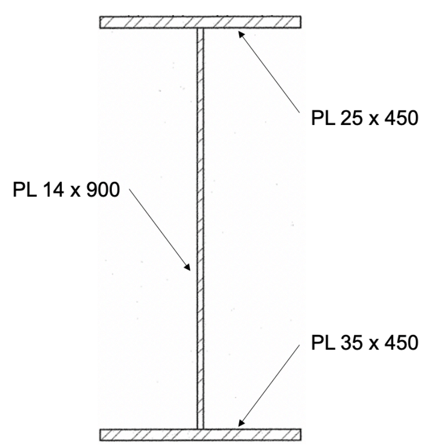

Figure 7-6: Steel girder cross section

7.2.2 Cast-in-Place Materials, DL_2

The weight of cast-in-place materials included the concrete deck tributary width (3.10 m girder spacing) and the concrete curb/parapet. For simplicity, a concrete curb and parapet were assumed uniform for all of the sample bridge drawings (900mm x

300mm), and it was assumed that the weight was evenly distributed to all of the girders. The density of reinforced concrete was assumed to be 25.5 kN/m^3 to include the mass of steel.

$$DL_2 = \rho_{RC} \times XS\ Area = 25.5\ \frac{kN}{m^3} \times \left[0.225\ m \times 3.10\ m + \frac{0.9\ m \times 0.3\ m \times 2}{5}\right] = 20.5\ \frac{kN}{m}$$

Equation 7-13

7.2.3 Wearing Surface, DW

The wearing surface thickness was 90 mm. For simplicity, the density of the asphalt wearing surface was assumed to be the same as concrete.

$$DW = \rho_{WS} \times XS\ Area = 25.5\ \frac{kN}{m^3} \times [0.09\ m \times 3.10\ m] = 7.1\ \frac{kN}{m}$$

Equation 7-14

The values from Equation 12, 13 and 14 were used to establish one set of the points required for the dead load regression lines. This process was repeated for the other available steel bridge drawings, and parameters were then developed to enable the estimation of the dead load effects for other span lengths within the 16-26 m range, as listed in Equation 15. Due to the minimal variation between the steel cross sections, an average wearing surface thickness of 85 mm and steel girder spacing of 2.45 m were assumed for the subsequent calculations.

$$DL_1 = 0.08\ \frac{kN}{m^2} \times span + 1.7\frac{kN}{m}$$

$$DL_2 = 0.07\ \frac{kN}{m^2} \times span + 17.3\frac{kN}{m}$$

$$DW = 5.3\ \frac{kN}{m}$$

Equation 7-15: Dead load regression equations for steel girders designed to the HSS-25 live load model as a function of span length

For the sample calculation steel bridge (span length of 24 m), the uniformly distributed dead loads based on the regression parameters are as follows:

$$DL_1 = \left(0.08\ \frac{kN}{m^2} \times 24\ m\right) + 1.7\frac{kN}{m} = 3.6\ \frac{kN}{m}$$

$$DL_2 = \left(0.07\ \frac{kN}{m^2} \times 24\ m\right) + 17.3\frac{kN}{m} = 19\frac{kN}{m}$$

$$DW = 5.3\ \frac{kN}{m}$$

Equation 7-16: Uniformly distributed steel girder dead loads estimated from the regression equations considering a span length of 24 m

These uniformly distributed loads were used to calculate the dead load moment and shear effects required for the reliability index calculation. The moment was calculated at midspan and the shear at the support, producing the values listed in Table 6. The bias factors used for calculation of the mean dead load effects are specified in the AASHTO LRFD calibration (Kulicki et al., 2007). A sample calculation for Factory-Made Materials is as follows:

$$Mean\ DL_1\ Moment\ Effect = 3.6\frac{kN}{m} \times \frac{24\ m^2}{8} \times 1.03 = 267\ kN.m$$

$$Mean\ DL_1\ Shear\ Effect = 3.6\frac{kN}{m} \times \frac{24\ m}{2} \times 1.03 = 44\ kN.m$$

Equation 7-17: Mean moment and shear effects for factory-made components (DL_1) considering a steel girder with a span length of 24 m

Table 7-6: Nominal and mean dead load effects and statistical parameters for a steel girder with a span length of 24 m

	Nominal Moment (kN.m)	Nominal Shear (kN)	Bias Factor	Mean Moment (kN.m)	Mean Shear (kN)	COV (Kulicki et al., 2007)
Factory-Made Materials, DL_1	259.2	43.2	1.03	267	44	0.08
Cast-in-Place Materials, DL_2	1368.0	228.0	1.05	1436	239	0.10
Wearing Surface, DW	381.6	63.6	1.0	382	64	0.25

7.3 NOMINAL RESISTANCE

The nominal resistance was calculated from the basic strength equation provided in AASHTO LRFD, listed as follows:

$$R_n = [1.25(DC) + 1.5(DW) + 1.75(LL\times IM)] \;/\; \phi$$

Equation 7-18

where DC is the dead load effect of structural components and non-structural attachments; DW is the dead load effect of the wearing surface and utilities; LL is the load effect resulting from the vehicular live load model (i.e., the HSS-25); IM is the dynamic load allowance, equal to 1.33; and ϕ is the resistance factor for each structural component reflecting its use (AASHTO, 2017).

The resistance factor, ϕ, for steel is specified in Clause 6.5.4.2 as 1.0 for both moment and shear (AASHTO, 2017).

The following nominal resistances can be determined using the dead load uniformly distributed loads calculated in Equation 16 and the live load effect produced by the HSS-25 live load model. The moment effects are calculated at midspan and the shear effects at the support.

$$R_{n,moment} = \frac{1.25 \times \left[\frac{(3.6+19)*24^2}{8}\right] + 1.5 \times \left[\frac{(5.3)*24^2}{8}\right] + 1.75 \times 1.33 \times 2563 \times 0.66}{1.0}$$

$$= 6541\ kN$$

$$R_{n,shear} = \frac{1.25 \times \left[\frac{(3.6+19)*24}{2}\right] + 1.5 \times \left[\frac{(5.3)*24}{2}\right] + 1.75 \times 1.33 \times 468 \times 0.82}{1.0}$$

$$= 1324\ kN$$

Equation 7-19: Nominal moment and shear resistances of a steel girder with a span length of 24 m based on the AASHTO LRFD design equation and the HSS-25 live load model

The mean resistances for moment and shear required for the reliability calculation are provided in Table 7. The bias factors used for calculation of the mean resistances are specified in the AASHTO LRFD calibration (Kulicki et al., 2007).

Table 7-7: Nominal and mean resistances and statistical parameters for a steel girder with a span length of 24 m

	Nominal Value	Bias Factor	Mean Value	COV
Bending Resistance, R_n	6541 kN.m	1.12	7326 kN.m	0.10
Shear Resistance, R_n	1324 kN	1.14	1509 kN	0.10

7.4 RELIABILITY ANALYSIS

For the reliability analysis of bridge members, the limit state function Z is expressed as R – S, where R corresponds to the resistance of the member and S is the total effect of the forces acting on it. The objective of the reliability analysis is to estimate the probability that R < S. Often the probability of failure is expressed through the reliability index, β, which is defined as:

$$\beta = -\Phi^{-1}(P_f)$$

Equation 7-20

where Φ^{-1} is the inverse of the standard normal distribution and P_f is the probability of failure.

The explicit form of the limit state function, Z, is as follows:

$$Z = R - S = R - [DL_1 + DL_2 + DW + (LL \times DF)]$$

Equation 7-21

The statistical properties for the random variables listed in Equation 21 have been previously calculated and provided in Tables 4 to 7.

7.4.1 Rackwitz and Fiessler First Order Reliability Method (FORM)

Similar to the calibration of the AASHTO LRFD code and the Canadian Highway Bridge Design Code, the method used to determine the reliability index in Chapter 4 was the iterative Rackwitz and Fiessler procedure. This method is based on normal approximations of non-normal distributions at a design point (R*, S*), which is the point of maximum probability on the failure boundary (i.e., when R*=S*). This procedure is outlined in several previous studies (CHBDC Calibration Subcommittee, 2002; Kulicki et al., 2007; Nowak, 1999).

The reliability indices calculated with the Rackwitz and Fiessler analytical method that consider the Lognormal resistance distributions, Normal dead load distributions, Gumbel Type 1 Manitoba live load distribution and Lognormal AASHTO LRFD live load distribution, were obtained using a FORM program written in R. The final reliability index values for a steel girder with a span of 24 m are listed in Table 8.

Table 7-8: Reliability index values for a steel girder with a span length of 24 m

	Bending	Shear
AASHTO LRFD Live Load Statistics	4.65	4.91
Manitoba Live Load Statistics	4.60	4.54

The following illustrates the reliability index calculation for the 24 m steel girder bridge described in this example, assuming that all of the distributions are normally distributed. In this simplified scenario, the following analytical solution applies:

$$\beta = \frac{(m_R - m_S)}{\sqrt{{\sigma_R}^2 + {\sigma_S}^2}}$$

Equation 7-22

where m_R is the mean value of the resistance, m_s is the mean value of the applied loads, and σ is the standard deviation. The standard deviation for the load and resistance parameters can be calculated with the mean values and the coefficient of variation. A summary of the mean values and standard deviations for moment and shear are provided in Tables 9 and 10.

Table 7-9: Summary of statistical parameters for a steel girder with a span length of 24 m in bending

	Mean Value (kN.m)	COV	Standard Deviation (kN.m)
Bending Resistance	7326	0.10	733
Factory-made Materials, DL_1	267	0.08	21
Cast-in-place Materials, DL_2	1436	0.10	144
Wearing Surface, DW	382	0.25	96
Manitoba LL	1999	0.10	200
AASHTO LRFD LL (HL-93)	1737	0.18	313

Table 7-10: Summary of statistical parameters for a steel girder with a span length of 24 m in shear

	Mean Value (kN)	COV	Standard Deviation (kN)
Shear Resistance	1509	0.10	151
Factory-made Materials, DL_1	44	0.08	3.5
Cast-in-place Materials, DL_2	239	0.10	24
Wearing Surface, DW	64	0.25	16
Manitoba LL	475	0.10	48
AASHTO LRFD LL (HL-93)	384	0.18	69

Using Equation 22, the reliability index can be approximated for independent normal distributions and compared to the values outputted in Table 8. A sample calculation for shear considering the Manitoba live load statistics is provided in Equation 23. The remaining estimated reliability indices are provided in Table 11.

$$\beta = \frac{(m_R - m_S)}{\sqrt{{\sigma_R}^2 + {\sigma_S}^2}} = \frac{1509 - (44 + 239 + 64 + 475)}{\sqrt{151^2 + 3.5^2 + 24^2 + 16^2 + 48^2}} = 4.27$$

Equation 7-23

Table 7-11: Simplified reliability index values assuming normal distributions

	Bending	Shear
AASHTO LRFD Live Load Statistics	4.30	4.62
Manitoba Live Load Statistics	4.16	4.27

The simplified reliability analysis produces similar values to those provided in Table 8, but smaller (larger probability of failure).

8.0 Appendix B: Mean Load and Resistance Values for the Reliability Analysis

Table 1 presents a summary of the statistical parameters and the representative distribution for the random variables considered for the reliability analysis. The majority of the bias factors and coefficients of variation are obtained from the AASHTO LRFD calibration, as referenced in the table.

Table 8-1: Summary of statistical parameters for the reliability analysis

Random Variable	Bias Factor	Coefficient of Variation	Distribution/ Reference
Bending Resistance Steel Members, Rn	1.12	0.10	Lognormal (Nowak, 1999)
Shear Resistance Steel Members, Rn	1.14	0.10	Lognormal (Nowak, 1999)
Bending Resistance Prestressed Concrete Members, Rn	1.05	0.07	Lognormal (Nowak, 1999)
Shear Resistance Prestressed Concrete Members, Rn	1.15	0.14	Lognormal (Nowak, 1999)
Factory-made Materials, DL_1	1.03	0.08	Normal (Nowak, 1999)
Cast-in-place Materials, DL_2	1.05	0.10	Normal (Nowak, 1999)
Wearing Surface, DW	1.00	0.25	Normal (Nowak, 1999)
Manitoba Live Load Effect, LL	1.10	0.10	Gumbel Type 1
AASHTO LRFD Live Load Effect, LL	1.18	0.18	Lognormal (Kulicki et al., 2007)
Distribution Factor, DF	1.0	-	Deterministic

The mean resistance and load effects for precast prestressed concrete box girders, precast prestressed concrete I-girders and steel girders for a span length range of 16 to 26 m are required for the reliability index value calculation. The mean values for the random variables, provided in Tables 2 to 7, are determined by multiplying the nominal values by the bias factor in Table 1. The standard deviation can be calculated by multiplying the mean value by the coefficient of variation in Table 1.

The sample calculation provided in Appendix A outlines how the nominal and mean values were obtained for a steel girder bridge with a span length of 24 m (note that truncations in the sample calculation result in slight differences between the sample calculation values and those listed in Tables 6 and 7).

8.1 PRECAST PRESTRESSED CONCRETE BOX GIRDERS

Table 8-2: Mean moment resistance and load effects [kN.m] for precast prestressed concrete box girders with a span length of 16 to 26 m

	16 m	18 m	20 m	22 m	24 m	26 m
Bending Resistance	1617.46	2035.36	2560.43	3161.48	3839.95	4625.58
Factory-made Materials, DL_1	376.90	517.88	689.81	895.73	1138.64	1421.59
Cast-in-place Materials, DL_2	0.03	19.43	88.60	185.39	313.67	477.32
Wearing Surface, DW	83.23	105.34	130.05	157.36	187.27	219.78
Manitoba LL	427.97	507.67	602.00	716.53	823.28	931.98
AASHTO LRFD LL (HL-93)	422.28	493.96	565.54	639.42	715.40	790.48

Table 8-3: Mean shear resistance and load effects [kN] for precast prestressed concrete box girders with a span length of 16 to 26 m

	16 m	18 m	20 m	22 m	24 m	26 m
Shear Resistance	755.46	820.66	896.86	977.30	1061.42	1150.43
Factory-made Materials, DL_1	94.22	115.08	137.96	162.86	189.77	218.71
Cast-in-place Materials, DL_2	0.01	4.32	17.72	33.71	52.28	73.43
Wearing Surface, DW	20.81	23.41	26.01	28.61	31.21	33.81
Manitoba LL	204.33	218.29	234.63	250.35	261.54	273.97
AASHTO LRFD LL (HL-93)	188.52	195.00	200.58	206.04	211.37	215.98

8.2 PRECAST PRESTRESSED CONCRETE I-GIRDERS

Table 8-4: Mean moment resistance and load effects [kN.m] for precast prestressed concrete I-girders with a span length of 16 to 26 m

	16 m	18 m	20 m	22 m	24 m	26 m
Bending Resistance	3612.54	4377.48	5189.28	6030.69	6884.70	7802.86
Factory-made Materials, DL_1	348.58	443.72	550.95	670.45	802.43	947.06
Cast-in-place Materials, DL_2	386.51	494.57	617.23	754.89	907.96	1076.83
Wearing Surface, DW	168.31	213.01	262.98	318.21	378.69	444.44
Manitoba LL	1029.51	1224.31	1455.15	1735.65	1998.14	2266.13
AASHTO LRFD LL (HL-93)	1015.83	1191.24	1367.01	1548.87	1736.32	1922.06

Table 8-5: Mean shear resistance and load effects [kN] for precast prestressed concrete I-girders with a span length of 16 to 26 m

	16 m	18 m	20 m	22 m	24 m	26 m
Shear Resistance	1352.41	1449.67	1536.59	1618.64	1694.73	1767.04
Factory-made Materials, DL_1	87.14	98.60	110.19	121.90	133.74	145.70
Cast-in-place Materials, DL_2	96.63	109.90	123.45	137.25	151.33	165.67
Wearing Surface, DW	42.08	47.34	52.60	57.86	63.12	68.37
Manitoba LL	355.11	383.87	416.95	449.15	473.34	499.81
AASHTO LRFD LL (HL-93)	327.63	342.90	356.45	369.66	382.54	394.03

8.3 STEEL GIRDERS

Table 8-6: Mean moment resistance and load effects [kN.m] for steel girders with a span length of 16 to 26 m

	16 m	18 m	20 m	22 m	24 m	26 m
Bending Resistance	3826.22	4641.70	5509.48	6411.66	7330.60	8321.79
Factory-made Materials, DL_1	99.74	133.05	172.68	219.13	272.91	334.52
Cast-in-place Materials, DL_2	619.82	790.55	983.52	1199.17	1437.95	1700.31
Wearing Surface, DW	168.31	213.01	262.98	318.21	378.69	444.44
Manitoba LL	1029.51	1224.31	1455.15	1735.65	1998.14	2266.13
AASHTO LRFD LL (HL-93)	1015.83	1191.24	1367.01	1548.87	1736.32	1922.06

Table 8-7: Mean shear resistance and load effects [kN] for steel girders with a span length of 16 to 26 m

	16 m	18 m	20 m	22 m	24 m	26 m
Shear Resistance	1199.68	1287.11	1365.66	1440.21	1509.77	1576.31
Factory-made Materials, DL_1	24.93	29.57	34.54	39.84	45.49	51.47
Cast-in-place Materials, DL_2	154.95	175.68	196.70	218.03	239.66	261.59
Wearing Surface, DW	42.08	47.34	52.60	57.86	63.12	68.37
Manitoba LL	355.11	383.87	416.95	449.15	473.34	499.81
AASHTO LRFD LL (HL-93)	327.63	342.90	356.45	369.66	382.54	394.03