DYNAMIC LOAD EFFECTS OF TRACKED AND WHEELED MILITARY VEHICLES FROM BRIDGE LOAD TESTING

AMPLIFICATION DYNAMIQUE DES VÉHICULES MILITAIRES À ROUES ET À CHENILLES LORS DU CHARGEMENT D'UN PONT

A Thesis Submitted to the Division of Graduate Studies
of the Royal Military College of Canada
by

Anthony Everitt, rmc
Captain

In Partial Fulfillment of the Requirements for the Degree of
Master of Applied Science in Civil Engineering

May 2019

For my sons Emmet and Sawyer –

may your wonder and desire to learn never end.

ACKNOWLEDGEMENT

I would like to extend a sincere thank you to my supervisors, Dr. Gordon Wight and Dr. Marc-André Dagenais, for their support, guidance, and encouragement throughout the whole process. Dr. Wight was on sabbatical at the time I commenced the program and always took the time to mentor and assist me in the early stages of this project. Their knowledge and resourcefulness were fundamental in the success of this research project, and contributed to its timely completion.

The RMC technicians, Mr. Dexter Gaskin, Mr. Steve Vanvolkingburgh, and Mr. Adam Watson, were crucial to the successful execution of field testing. They all spent a considerable amount of time away from their families in order to facilitate reconnaissance, site set-up, instrumentation of the bridge, and field-testing. Their expertise and years of experience contributed to the success of such a complex project. The field-testing would not have been possible without their involvement.

The logistics involved in re-routing a tank through Petawawa in order to facilitate this testing required the involvement of many stakeholders in the process. Director Armament Sustainment Program Management (DASPM) assisted early on in the planning, and provided support and funding for the movement of a Leopard 2 tank, specifically Mr. Henry Litjens, the Equipment Support Manager – Leopard 2 Fleet, and Mr. Daniel Robichaud, the DASPM 4 Equipment Management Team Coordinator. The G34 of the Canadian Army (CA) was filled by three different individuals during this process, all of whom continued to push for and support the conduct of this testing. The Commandant of RMC was engaged and supported this initiative to the Chief of Staff Army Operations in order to provide the necessary approvals and support for this project. The support from all was critical for this research project to proceed as planned. Undoubtedly, many more individuals assisted along the way in the countless hours of staff work required to support this endeavour, and the effort from all is extremely appreciated.

This project was made possible with the funding support from Director Combat Support Equipment Management (DCSEM) 3. DCSEM 3 is currently sponsoring improvements and developments to the Military Load Classification (MLC) Software Suite, including investigation into some of the existing MLC doctrine issues.

On site in Petawawa, members of 2 Combat Engineer Regiment (CER) supported the instrumentation of the Mountbatten Bridge, as well as the conduct of the field-testing portion. Eager sappers assisted with the instrumentation of the bridge and contributed to it being completed ahead of schedule. 2 CER enthusiastically provided increased support for the field-testing portion and increased the number of wheeled military vehicles available for the testing from one to three. Early involvement as G34 CA and then as Commanding Officer of 2 CER during the conduct of the field-testing, Lieutenant-Colonel Chantal Tetreault provided guidance and behind the scenes support for this initiative which is particularly appreciated.

Members of the US Army Engineer Research and Development Center also supported this research and were present for the testing. Remisz Consulting Engineers Ltd. was extremely helpful with providing reports and initial drawings of the Mountbatten Bridge.

An exceptional thank you is extended to my wife, Chelsey, who encouraged and supported me throughout the completion of my Master's Degree. The busy life with a toddler, Emmet, became increasingly busy with the arrival of our second son Sawyer in June 2018. Chelsey supported me with the time needed to complete my studies, and held down the fort while I was away. She would always find the time to provide constructive comments and critiques on my work, which was a considerable benefit.

ABSTRACT

The perceived and observed differences in the dynamic behaviour between wheeled and tracked military vehicles should be accounted for in the application of appropriate dynamic load effect values for bridge design and assessment. No current bridge design or assessment code provides guidance on methods to differentiate between the dynamic loading effects of wheeled and tracked vehicles. Depending on the code or guideline being applied for analysis, these dynamic loading effect values may be represented and referred to as Dynamic Load Allowance (DLA), Dynamic Amplification Factor (DAF), or Dynamic Impact Factors (IM). The North Atlantic Treaty Organization (NATO) utilizes a Military Load Classification (MLC) System to compare the load effects of vehicles to the capacity of bridges in order to determine the feasibility of crossing. The MLC System uses the same dynamic loading effects values, for both wheeled and tracked vehicles, which can significantly limit the mobility of tracked vehicles, particularly main battle tanks, on military operations.

Bridge load testing was carried out on a continuous two-span steel girder composite bridge to compare the dynamic loading effects between three wheeled military vehicles and Canada's main battle tank, the Leopard 2. Ninety different load tests were completed on a fully instrumented bridge. The dynamic effects were then used to calculate appropriate DLA values for individual vehicles and per vehicle category (wheeled and tracked) using various combinations of recommended design code reliability indexes and live-load factors. Results indicate that it may be appropriate to reduce the DLA used for military tracked vehicles by one-third of that used for military wheeled vehicle analysis. A review of several nations' DLA values was carried out, and application of a reduced DLA for tracked vehicles could result in an increase to predicted bridge capacity of 5% to 13% for main battle tanks.

Surface irregularities are seldom accounted for in bridge design or assessment codes, yet research indicates a significant increase in dynamic effects can occur when a vehicle travels over an obstacle. These obstacles or surface irregularities could be encountered during military conflict or post natural disaster situations. A comprehensive test program was carried with the Leopard 2 in order to examine the dynamic loading increase when it travelled over obstacles, and a marked increase in the tank's dynamic loading effects was noted. A similar yet reduced test program was carried out using wheeled vehicles, and those results demonstrated a considerable increase in dynamic loading effects when compared to the Leopard 2. In some instances, the dynamic loading effects generated by wheeled vehicles were approximately five times that of the Leopard 2. All vehicles also carried out a series of tests in which they decelerated from various speeds to a full stop as rapidly as possible. Design codes generally apply a braking force separately from the DLA, and longitudinally at or near the bridge deck. However, this research indicates an increase in dynamic effects when braking occurs, particularly over short distances.

Due to the different dynamic behaviour between wheeled and tracked vehicles, the application of DLA during bridge capacity analysis should be differentiated between vehicle types. A reduced DLA for tracked vehicles has the possibility to significantly increase the mobility of military forces particularly when combined with appropriate live-load factors for main battle tanks.

RÉSUMÉ

La différence entre l'effet dynamique des véhicules militaires à roues et à chenilles doit être prise en compte lors de l'application des coefficients de majoration dynamique pour la conception et l'évaluation de ponts. Aucun code actuel sur la conception ou l'évaluation de ponts ne fournit des directives permettant de faire la distinction entre les véhicules à roues et chenilles. Selon le code ou la norme utilisée aux fins d'analyse, différentes terminologies sont utilisées pour référé à l'effet dynamique des charges : coefficient de majoration dynamique, coefficient d'amplification dynamique ou encore, facteur d'impact dynamique. L'Organisation du traité de l'Atlantique Nord (OTAN) utilise un système de classification militaire des charges (CMC) pour comparer l'effet des charges des véhicules à la capacité du pont afin de déterminer la possibilité de franchir le pont. Le système CMC utilise les mêmes valeurs de coefficient de majoration dynamique pour les véhicules à roues et à chenilles, ce qui peut avoir une incidence considérable et limiter la mobilité des véhicules à chenilles, en particulier pour les chars de combat dans le cadre d'opérations militaires.

Un programme expérimental a été mené sur un pont continu multi poutre en acier composite de deux travées afin de comparer l'effet dynamique de trois véhicules militaires à roue ainsi qu'un véhicule à chenilles, le char d'assaut principal du Canada, le Léopard 2. Un total de quatre-vingt-dix essais ont été effectués sur le pont entièrement instrumenté. Les effets dynamiques mesurés ont ensuite été utilisés pour calculer les valeurs de coefficient de majoration dynamique appropriées pour chaque véhicule et par catégories de véhicule (à roues et à chenilles) à l'aide de diverses combinaisons d'index de fiabilité provenant de norme de conception et de facteurs de surcharge recommandés. Les résultats indiquent que le coefficient de majoration dynamique des véhicules militaires à chenilles pourrait être réduit du tiers de ceux utilisés pour l'analyse des véhicules militaires à roues. De plus, une compilation des coefficients de majoration dynamique utilisée dans différents pays a été effectué et l'application d'une valeur réduite de ce coefficient pour les véhicules à chenilles pourrait augmenter la capacité prédite des ponts de 5% à 13% pour les chars de combat.

Les irrégularités de surface sont rarement prises en considération par les codes relatifs à la conception et à l'évaluation des ponts. Cependant, les recherches indiquent qu'une importante augmentation des effets dynamiques peut se produire lorsqu'un véhicule franchit un obstacle. De tels obstacles ou de telles irrégularités de surface pourraient se trouver sur un pont stratégique dans le cadre d'un conflit militaire ou à la suite de catastrophes naturelles. Un programme expérimental a été effectué avec le Léopard 2 dans le but d'examiner l'augmentation de l'effet dynamique lors du franchissement d'obstacles, et une légère augmentation des effets de charge dynamique a été constatée. Par la suite, un programme d'essais similaire a été mené avec des véhicules à roues, et les résultats ont démontré une augmentation significative des effets dynamiques des charges comparativement au Léopard 2. Dans certains cas, les effets dynamiques des charges produits par les véhicules à roues se sont avérés environ cinq fois plus importants que le Léopard 2. Tous les véhicules ont aussi subi une série d'essais lors desquels ils ont décéléré à partir de diverses vitesses jusqu'à l'arrêt complet aussi rapidement que possible. Les normes de conception appliquent une force de freinage séparément du coefficient de majoration dynamique, longitudinalement au niveau du tablier de pont ou près de celui-ci. Cette étude démontre une augmentation de l'effet dynamique des charges lors d'un freinage, en particulier sur de courtes distances.

En raison des différences entre le comportement dynamique des véhicules à roues et à chenilles, l'utilisation d'un coefficient de majoration dynamique propre à chaque type de véhicules devrait être utilisée lors de l'analyse d'un pont. L'utilisation d'un coefficient de majoration dynamique réduit pour les véhicules à chenilles pourrait augmenter considérablement la mobilité des forces militaires, en particulier lorsqu'elle est combinée aux facteurs de surcharge appropriés pour les chars de combat.

CO-AUTHORSHIP STATEMENT

This thesis has been written in the article-based format as laid out in the Royal Military College of Canada Thesis Preparation Guidelines. The author of this thesis, Captain Anthony Everitt, was the main contributor to both articles, with the co-authors providing guidance, advice and feedback. As the author plans to submit both articles for publication in peer-reviewed journals, the individual articles will include both supervisors as co-authors.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
BEF	Braking Effects Factor
CA	Canadian Army
CABMLC	Canadian Analytical Bridge Military Load Classification
CAF	Canadian Armed Forces
CAVMLC	Canadian Vehicle Military Load Classification
CDSB	Canadian Division Support Base
CER	Combat Engineer Regiment
CHBDC	Canadian Highway Bridge Design Code
CORVMLC	Correlation and Vehicle Military Load Classification
CSA	Canadian Standards Association Group
DA	Dynamic Amplification
DAF	Dynamic Amplification Factor
DAQ	Data Acquisition
DASPM	Director Armament Sustainment Program Management
DCSEM	Director Combat Support Equipment Management
DEF	Dynamic Effects Factor
DIC	Digital Image Correlation
DLA	Dynamic Load Allowance
DLF	Dynamic Load Factor
E-LAV	Engineer Light Armoured Vehicle
EROC	Expedient Route Opening Capability
EU	European Union
FLS	Fatigue Limit State
GVW	Gross Vehicle Weight
HLVW	Heavy Logistics Vehicle Wheeled
IM	Impact Factor
ISC	Infantry Section Carrier
L1	Location 1
L2	Location 2
L3	Location 3
LRFD	Load and Resistance Factor Design
LSD	Limit State Design
MBT	Main Battle Tank

MLC	Military Load Classification
NATO	North Atlantic Treaty Organization
NCHRP	National Cooperative Highway Research Program
NG	New Generation
OHBDC	Ontario Highway Bridge Design Code
PA	Permit – Annual or Project
PB	Permit – Bulk Haul
PC	Permit – Controlled
PL	Plate
PS	Permit – Single Trip
RFBMLC	Rapid Field Bridge Military Load Classification
RMC	Royal Military College of Canada
SLS	Serviceability Limit State
STANAG	Standardization Agreement
T-F	Tracked Fighting
T-T	Tracked Transport
ULS	Ultimate Limit State
USA	United States of America
W-F	Wheeled Fighting
W-T	Wheeled Transport
WWF	Welded Wide-Flange

LIST OF SYMBOLS

D	Transverse direction distance between reference point and the wheel line
$DEF_{Obstacle}$	Estimated DEF Value for a tracked vehicle travelling over an obstacle
F	Fundamental Frequency
H	Bridge Superstructure Height or Depth
h	Obstacle Height in millimeters
Hz	Hertz
I	Impact Fraction
in	Inch
kg	Kilogram
km/h	Kilometers per hour
kN	Kilo Newton
L	Span Length
m	Metre
mm	Millimetre
MPa	Mega-Pascal
R_{Brake}	Braking Response
R_{Dyn}	Dynamic Response
R_{Sta}	Static Response
s	Separation Factor
t	Tonnes
v	Coefficient of Variability
W	Half-Width of a Vehicle
α	Relative position parameter with respect to a reference point
α_L	Live-Load Factor
β	Reliability Index
ε	Strain
δ	Bias Coefficient
μ	Micro
σ	Standard Deviation
$^\circ$	Degree
%	Percent

1. INTRODUCTION

1.1 Project Background

Bridges are necessary infrastructure located throughout the world that provide access from one side of a waterway, road, or obstacle to the other side [1]. As of 2016, there were over 47,000 publicly owned bridges in Canada [2]; as such, their use in everyday life is prevalent. From a military perspective, a critical task completed by military commanders when planning missions and operations is route identification and selection. A key component of route selection is knowing which bridges can support the involved vehicle traffic.

Bridge capacities are typically assessed by bridge engineers using national codes [3]. Using approaches outlined in these codes, bridge engineers employ models of varying complexity to assess bridge structures. These national codes are applied by host nations for movements during peacetime, and bridges on military installations are typically assessed using these codes. When military forces are on operations, military engineers apply simplified methods of estimating bridge capacity based on information that can be rapidly assessed or estimated. The North Atlantic Treaty Organization (NATO) utilizes the Military Load Classification (MLC) System outlined in NATO Standardization Agreement (STANAG) 2021 Edition-8 to calculate the MLC of bridges and vehicles [4]. MLC values for vehicles are determined by calculating the governing shear and moment loading effects of a convoy of that vehicle type, on spans ranging from 1 m to 100 m, and these results are then compared to the NATO standard theoretical military vehicles (wheeled and tracked). Similarly, MLC capacity ratings for bridges are calculated by determining the allowable shear and moment capacity values, which are then converted to an MLC value, representative of its military load capacity. A vehicle MLC rating must be lower than that of a bridge in order to safely cross. These calculations can be accomplished through hand calculations or with several different software tools designed for MLC calculations [5] [6] [7] [8].

To safely assess a bridge's capacity, whether using a national bridge code or a NATO MLC System approach, factors are applied to decrease the resistance of the bridge and increase the effects of the load. These adjustments account for a variety of issues including the variability of material properties, quality of construction, and the actual effects of traffic. One load factor, the Dynamic Load Allowance (DLA), relates to the dynamic effects that vehicles exert on bridges. The MLC System and national codes currently uses the same load factors when assessing capacity for both wheeled and tracked vehicles. The use of the same DLA for wheeled and tracked vehicles may be inappropriate due to the different dynamic behavior typically perceived between wheeled and tracked vehicles. A reduced DLA for tracked vehicles would greatly increase the mobility of tanks at home on exercises and abroad on military operations. In order to validate the different dynamic behavior between wheeled and tracked vehicles, experimentation was carried out by means of a bridge load test with Canada's main battle tank (MBT), the Leopard 2, and three different wheeled vehicles. The data from the testing was used to calculate the dynamic effects on the bridge for each vehicle, which then underwent a code calibration process in order to compare the vehicles' performance against existing DLA values.

Military forces on operations may encounter bridges that have had deliberate obstacles placed across them, or bridges that have been damaged. Little guidance exists for understanding the dynamic effects that surface irregularities, be they from obstacles or damage, can produce. Understanding the increased magnitude of dynamic effects that surface irregularities can cause is important knowledge for operators of military vehicles so that they are aware of the effects that they can generate.

In order to investigate these issues, a suitable bridge for testing was identified at 4th Canadian Support Base (CDSB) Petawawa, logistic coordination for all involved assets was carried out, various instrumentation was installed on the bridge, and a two-day program involving a tracked fighting vehicle and three different wheeled vehicles was completed.

1.2 Aim

The aim of this research project was primarily to validate the different dynamic behaviour and effects between wheeled and tracked vehicles on bridges. Testing with obstacles and braking was used to examine the increase that these events can cause on the flexural stresses in bridges. This research project aims to validate and quantify the different dynamic behaviour between tracked and wheeled military vehicles in order to develop a suggested reduced DLA for tracked vehicles, and to examine the dynamic effects generated by wheeled and tracked military vehicles braking, and passing over surface irregularities on a bridge.

1.3 Scope

The scope of this project was constrained to examine the dynamic effects of Canada's MBT, to compare the dynamic effects between wheeled and tracked military vehicles, to provide a suggested reduced DLA for tracked vehicles, and to examine the effects of surface irregularities and braking on dynamic effects. The test program involved static load tests for all vehicles, tests over a smooth surface both along the centerline and the edge of the bridge, tests over a series of obstacles to simulate surface irregularities, and a series of brake check tests. The tracked fighting vehicle was able to complete a more comprehensive test program than the wheeled vehicles. The Mountbatten Bridge (shown in Figure 1-1), located at 4 CDSB Petawawa in Petawawa, Ontario was selected as a suitable bridge to use for this research project. The Royal Engineers of the British Army constructed the Mountbatten Bridge in the years 1977 and 1978. The bridge is a continuous structure with two spans of 29.6 m and 32.9 m, and is a steel stringer with composite connection to a concrete deck construction. The bridge is 7.5 meters wide overall, with a 6-meter roadway width.

Figure 1-1. The Mountbatten Bridge in Winter.

Instrumentation installed on the Mountbatten Bridge included strain gauges, wired linear position string potentiometers, accelerometers, and a stochastic pattern for Digital Image

Correlation (DIC). In total, there were 96 data channels sampling at 1200 Hz. While additional instrumentation was installed on the test bridge, not all data recorded is necessary for the calculation of the dynamic effects presented in this document.

The testing was conducted over two days, with the first day focussing on the tracked vehicle, the Leopard 2 shown in Figure 1-2, and the second day for the three different wheeled vehicles. The wheeled vehicles used in testing were: A Heavy Logistics Vehicle Wheeled (HLVW) with a low-bed, an Engineer Light Armoured Vehicle (E-LAV), and an Expedient Route Opening Capability (EROC) vehicle know as the Cougar. The road was graded between test days to facilitate similar approach conditions for all tests.

Figure 1-2. Canada's Main Battle Tank, the Leopard 2.

The tracked fighting vehicle was able to complete a more comprehensive test program than the wheeled vehicles. All vehicles' static responses were obtained along the centerline in both directions, and along the edge of the bridge. The complete test program carried out is shown in Table 1-1. The smooth surface test results generated Dynamic Effects Factor (DEF) values for all vehicles from three principal instrumented locations. The smooth surface DEF values then underwent a calibration process in order to develop DLA values, which were then compared to each other and against other design code provisions for DLA. The obstacle tests were used to simulate surface irregularities and further examine the difference between wheeled and tracked vehicles' dynamic behaviour. The braking effects of all vehicles were also examined to explore the relationship between braking and observed dynamic effects, as well as to demonstrate how stopping distance can influence the observed dynamic effects.

Table 1-1. Complete Test Program.

Tests	10		20		30		40		50	
Centerline – Smooth Surface	T	W	T	W	T	W	T	W	T	W
Edge – Smooth Surface	T	W	T	W	T	W	T	W	T	W
Centerline with Single High Obstacle	T		T		T		T		T	
Centerline with Double High Obstacle	T	W	T	W	T	W	T		T	
Edge with Double High Obstacle	T		T		T		T		T	
Centerline with One Triple High Obstacle	T				T				T	
Brake Check	T	W			T	W			T	

Vehicle Speed (km/h) applies to columns 10–50.

Note: T represents tests carried out by the tracked vehicle, the Leopard 2.
W represents tests carried out by all three wheeled vehicles.

A total of 90 different tests were carried out between the four vehicles. The research team had complete control of the bridge and approaches during the days of testing. Testing resulted in large quantities of data, not all of which were used in the development of this document. The intent is to examine additional aspects from the conduct of a bridge load testing that were not included in the scope of this document for additional Journal publications.

1.4 Thesis Organization

This thesis is written in manuscript/article-based format as detailed in the Royal Military College of Canada (RMC) Thesis Preparation Guidelines [9] and is comprised of five chapters. Chapter 1 encompasses the project introduction, and outlines the background, aim, and scope of the research. Chapter 2 is the literature review portion which provides an in-depth review of past and relevant literature pertaining to this thesis. Chapter 3 is a standalone article that will be submitted to a suitable engineering journal for publication. The article in Chapter 3 relates to conducting smooth surface tests with a tracked vehicle and three different wheeled military vehicles in order to compare their distinctly different dynamic behaviour, and provide a suggested reduced DLA value for tracked vehicles. Chapter 4 is another standalone article that will also be submitted to a suitable engineering journal for publication. The second article includes some similar information related to the test set-up and procedures. Results and discussion in the second article focus on how vehicles dynamic loading effects are increased by surface irregularities, and examine the effects of braking on the dynamic loading effects generated by vehicles. Chapter 5 provides a summary of the research project and discusses recommendations for future work. A series of Appendices follow Chapter 5.

Chapter 3 and Chapter 4 contain their own reference lists, individually numbered, as they have the potential to be standalone documents. All references used for Chapters 1, 2, 5 and the appendices are located at the end of this thesis and are numbered independently from Chapters 3 and 4.

1.5 Description of Appendices

Pertinent information related to each manuscript was included in their respective chapters. Other information relevant to the conduct of this research is provided in the appendices that are located after Chapter 5, at the end of this document. General information as to the conduct of testing was included in each manuscript, while the appendices outline the preparatory work that was required to successfully carry out load testing of a bridge.

Appendix A contains details of the instrumentation of the Mountbatten Bridge. It also includes details on the preliminary analysis that was conducted in order to determine the ideal location for instrumentation, further information on the locations for all instruments, details on the numbering nomenclature used, and some site photographs of set-up and instrumentation. Appendix A should be of use to anyone who plans to conduct load testing of a bridge as it would assist them in identifying some of the aspects that should be considered.

Appendix B covers information and photographs of the platform used to install instrumentation on the Mountbatten Bridge. Because the bridge spanned a waterway, it was necessary to have a method to provide under bridge access in order to install the various instrumentation.

Appendix C through E contain data output that was used to calculate DEF values. Due the large quantity of date, Appendix C through E only report the maximum observed strain values at pertinent locations and instrumentation depending on the test performed. Appendix C is focussed on the smooth surface tests that looks at data from the three primary instrumented locations. Appendix D provides the data from the tests that included lumber as obstacles/surface irregularities. Appendix E is the results from the braking tests.

Appendix F contains RESTRICTED information that will only be contained in a limited distribution copy of the document and not in the online publication. The Appendix contains loading and dimensions for the test vehicles.

2. LITERATURE REVIEW

2.1 General

This section provides an in-depth review of past and relevant literature pertaining to the Military Load Classification (MLC) System, load factors, the Dynamic Load Allowance (DLA), surface irregularities, and braking effects. It also provides the reader with sufficient background knowledge of instrumentation and bridge testing procedures to provide a basic understanding of the field testing performed in support of this research. This literature review encompasses all topics covered in the manuscripts that are included within this document, serving as a review for the entire project. Each individual manuscript has its own brief literature review which summarizes the most important information from this review to provide the manuscript reader the required and most pertinent background knowledge. References for the complete thesis and this Chapter can be found after Chapter 5, while Chapters 3 and 4 have their own reference lists as they are stand-alone papers.

2.2 Military Load Classification System

Route identification and selection is a critical task completed by commanders when planning missions and operations. A key component of route selection is knowing which bridges can support the involved vehicle traffic. Furthermore, when militaries conduct exercises or operations, at home or abroad, military vehicles will undoubtedly be required to cross civilian bridges, and it is essential that loading effects and bridge capacities are understood and quantified. The North Atlantic Treaty Organization (NATO) utilizes the MLC System outlined in NATO Standardization Agreement (STANAG) 2021 Edition 8 to calculate the MLC of bridges and vehicles. The aim of STANAG 2021 is to provide NATO forces with a standard method for computing the MLC representing the load capacities of all bridges, military ferries and rafts, and the loading effects of military vehicles [4]. It is important to note that the MLC value is only a number and does not represent the mass of the vehicle [4], or a tonnage capacity value for a bridge.

The MLC system uses 32 hypothetical vehicles (16 wheeled and 16 tracked) (see Figure 2-1 for an excerpt), that are based off of representative NATO nations vehicles, to create unfactored graphs of their unit bending moments and shear effects on bridge spans ranging from 1 m to 100 m [4]. These charts and tables are generated by the hypothetical MLC vehicles from MLC 4 through MLC 150 [4]. Different curves are derived for wheeled and tracked vehicles and a representative set of curves for the shear effects of theoretical wheeled vehicles is shown in Figure 2-2, for the unit bending moment of theoretical wheeled vehicles is shown in Figure 2-3, and for the unit bending moment of theoretical tracked vehicles is shown in Figure 2-4.

MLC values for vehicles are determined by calculating the governing shear and moment loading effects of that vehicle, on spans ranging from 1 m to 100 m, and these results are then compared to the theoretical vehicles. Vehicles are assumed to be travelling in convoys with 30.5 m spacing (100 feet) between the trailing and leading axles of consecutive vehicles [4], therefore the effects of multiple vehicles are considered in spans greater than 30.5 m. The maximum shear and moment values are then compared to the theoretical vehicle load effects; and whichever span length yielded the highest MLC value, will govern the MLC value for that vehicle.

The benefit of analyzing vehicles through this method, versus using its number of axles and Gross Vehicle Weight (GVW), is that it permits each vehicle to be rated on the nominal maximum effects that it generates on a simple span [10]. MLC values should be calculated when vehicles are fully loaded with fuel, ammunition, personnel, kit, load, etc. This may in turn limit the mobility of some vehicles, as they may not often be operating at a fully laden condition.

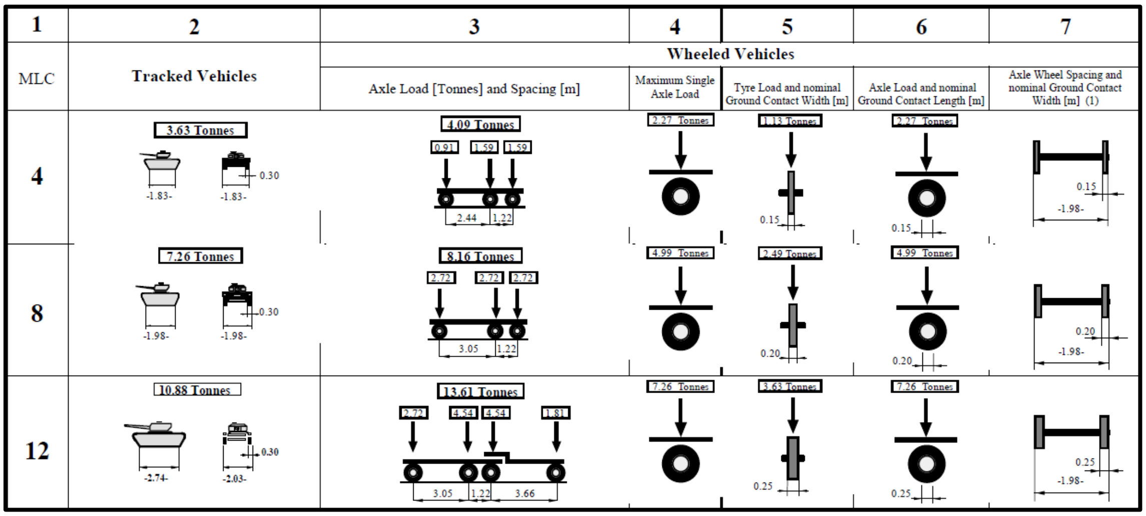

Figure 2-1. Excerpt of Hypothetical MLC Vehicles [4].

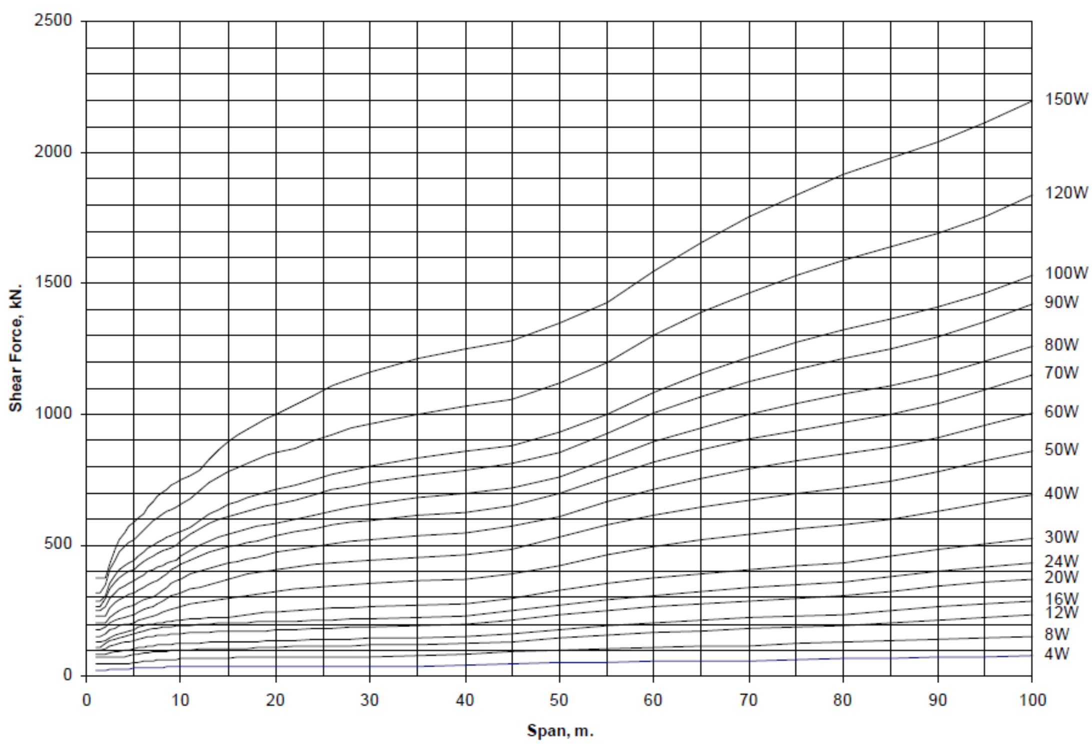

Figure 2-2. Shear Forces due to Theoretical Wheeled Vehicles - Spans from 1 m to 100 m [11].

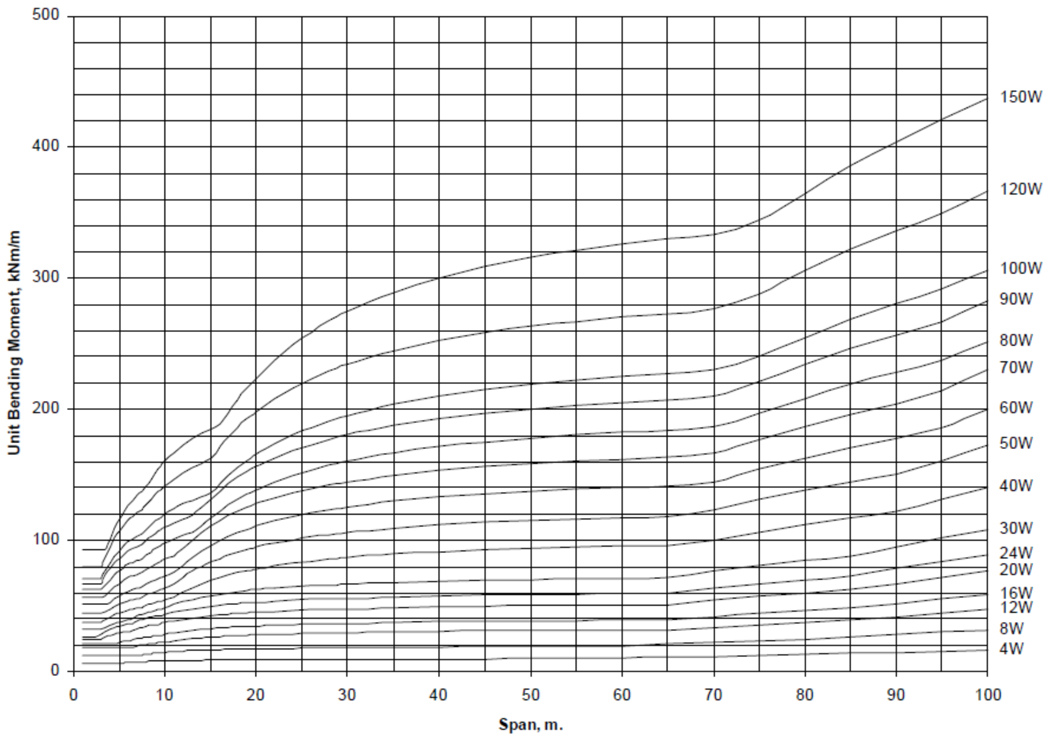

Figure 2-3. Unit Bending Moments due to Theoretical Wheeled Vehicles - Spans from 1 m to 100 m [11].

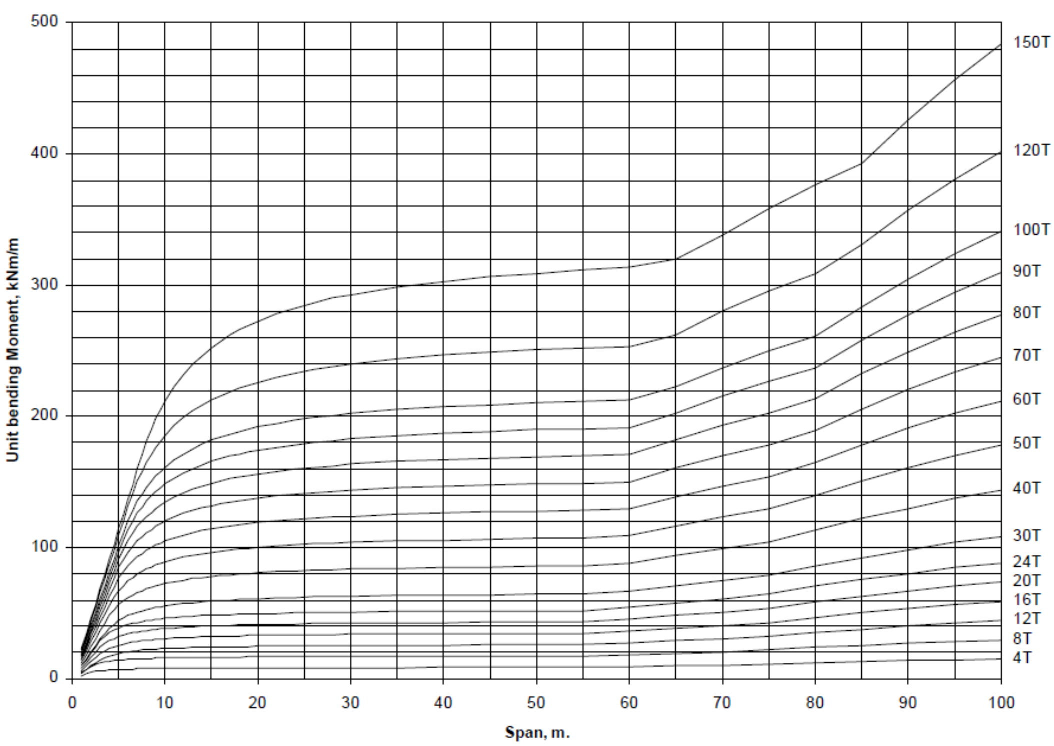

Figure 2-4. Unit Bending Moments due to Theoretical Tracked Vehicles - Spans from 1 m to 100 m [11].

To assign an MLC capacity rating to a bridge, it is necessary to determine the allowable shear and moment capacity of that structure. Once the allowable shear and moment values are determined, their values are plotted using the bridge span on the graphs similar to those seen in Figure 2-2 and Figure 2-3 to determine the MLC capacity rating for that bridge. Because the load effects of wheeled and tracked military vehicles are unique and may be different from normal civilian traffic [10], it is necessary to appropriately account for this in determining MLC capacities of bridges.

The MLC system described in STANAG 2021 also establishes three crossing conditions of Normal, Caution, and Risk Crossing. Normal Crossing of a bridge permits unrestricted use of civilian bridges and normal accepted restrictions for any military bridges. Tactical or emergency situations may arise were higher MLC vehicles are required to cross certain bridges. With more controlled crossing criteria and/or the use of less stringent safety criteria, a Caution or Risk Crossing may be followed. A Caution Crossing involves vehicles driving guided along the centerline of the bridge, with a speed not exceeding 5 km/h, and braking, accelerating, or changing gears prohibited. For analysis of this situation, the same safety factors will apply as a Normal Crossing, except no DLA will be used in the capacity calculations. The vehicle crossing constraints remain the same for Risk Crossing, with all safety factors being revised. The increased probability of bridge failure in a Risk Crossing is accepted, along with the understanding that structural stresses may be near the yield limits, but should not exceed the ultimate limit [4].

Military doctrine establishes hand analytical methods to follow for the MLC calculations of vehicles and bridges. Military doctrine prescribes certain load factors to use in calculations, similarly to situations where the load capacity of a bridge was determined using civilian design codes. Software has been developed to aid in MLC calculations. The Canadian Windows MLC software suite is comprised of four different software. Two analytical methods exist: Canadian Analytical Bridge Military Load Classification (CABMLC) [5] and Rapid Field Bridge Military Load Classification (RFBMLC) [6]. CABMLC follows a thorough analytical process, while RFBMLC assumes some analytical information in order to simplify the input. A traffic correlation method software, Correlation and Vehicle Military Load Classification (CORVMLC) [7], exists that uses individual country's historical design vehicle information to estimate MLC values for bridges, as well as it provides the user the ability to estimate the MLC of vehicles. There is also the Canadian Vehicle Military Load Classification (CAVMLC) software that is dedicated to estimating the MLC of vehicles [8]. The German software is BRASSCO-New Generation (NG) [12] that follows a simplified and rapid analytical process similar to RFBMLC. The hand analytical and software versions differentiate between wheeled and tracked military traffic in their final MLC determination, yet the load factors used are identical for wheeled and tracked vehicle assessment.

2.3 Load Factors

The live-load capacity of a bridge can be estimated by determining the resistance of the structure and then subtracting the effects of the dead load. To safely assess a bridge's capacity, factors are applied to decrease the resistance of the bridge and increase the effects of the loads. These adjustments account for a variety of issues including the variability of material properties, quality of construction, and the actual effects of traffic and are typically determined using reliability-based methods at the ultimate limit state. The live-load factor specifically accounts for unknown overloads during a bridge's lifetime and uncertainties in the load analysis [13]. Although

STANAG 2021 is established to provide NATO forces with a standard method for computing MLCs, no specific method is imposed and each country is permitted to use their own procedures and safety factors [4] [14]. Therefore, STANAG 2021 has moved towards the development of a safety concept while allowing individual countries to follow their civilian codes while establishing national standards [4]. STANAG 2021 has suggested the use of a Safety Factor, or Reliability Index, β, of 3.3 for Normal Crossings in bridge classification [4]. This allows NATO nations to apply their national standards from their respective codes, for example the Canadian S6-14 Canadian Highway Bridge Design Code (CHBDC), or the American Association of State Highway and Transportation Officials (AASHTO) Manual for Bridge Evaluation, or the Eurocode. Within these standards, there is a dearth of guidance on assessing the load effects generated by tracked vehicles, be they civilian or military.

The use of Reliability Indexes, β, is prevalent in many nations' codes and is the recommended basis for all current Limit State Design (LSD) codes [13]. The Reliability Index is selected in order to provide an expected structural behaviour level of safety, which is inversely related to its notional probability of failure. The relationship between Reliability Index, β, and the Notional Probability of Failure is shown in Table 2-1. As per *CSA S6-14 CHBDC*, new structures are required to achieve a target β value of 3.75 for a 75 year design life, while bridge assessment carries different suggested β values depending on the level of inspection and details of the structural system of the bridge [3].

Table 2-1. Relationship between Reliability Index, β, and Probability of Failure [13].

Reliability Index, β	Notional Probability of Failure
2.00	2.3×10^{-2} or 1:44
2.25	1.2×10^{-2} or 1:81
2.50	6.2×10^{-3} or 1:160
2.75	2.8×10^{-3} or 1:360
3.00	1.4×10^{-3} or 1:740
3.25	5.6×10^{-4} or 1:1 800
3.50	2.3×10^{-4} or 1:4 300
3.75	8.8×10^{-5} or 1:11 000
4.00	3.2×10^{-5} or 1:31 500
4.25	1.1×10^{-5} or 1:93 500
4.50	3.4×10^{-6} or 1:294 000

The *CSA S6-14 CHBDC* used LSD philosophy in applying load and resistance factors when designing structures [3]. A structure, e.g. a bridge, should be designed so that when at the ultimate limit state, the factored resistance exceeds the total factored load effect [3]. All structural components are to comply with the ultimate limit state (ULS), the serviceability limit state (SLS), and the fatigue limit state (FLS) provisions outlined in the *CSA S6-14 CHBDC* [3]. It is important to note that many current bridge codes are primarily concerned with the design of new structures; whereas assessment and classification carries an associated uncertainty [4]. As such, many assessments indicate a reduced performance for existing infrastructure when modern design loading is applied [15].

The *CSA S6-14 CHBDC* provides load combinations requiring consideration when assessing the FLS, SLS, or ULS of a structure. The load combinations fit into three categories of permanent loads, transitory loads, and exceptional loads [3]. Included in the transitory loads category are live-load factors, which include the DLA when applicable [3]. *CSA S6-14 CHBDC* establishes a DLA of a 0.25 for most spans and a live-load factor based on the Reliability Index, β [3]. STANAG 2021 suggests a live-load factor of 1.35 in MLC for both wheeled and tracked vehicles under normal crossing conditions; however, there is no suggested DLA value and it is recommended to follow individual country standard practice depending on vehicle type and expected use of the bridge [4]. Canadian and United States doctrine suggest a DLA of 0.15 for MLC application [11] [16].

CSA S6-14 CHBDC uses a Normal traffic category and four classes of permit vehicles. The four classes of permit vehicles are: (1) Annual or Project (PA); (2) Bulk Haul (PB); (3) Controlled (PC); and Single Trip (PS). PA traffic is explained as:

> "PA traffic shall include the vehicles authorized by permit on an annual basis or for the duration of a specific project to carry an indivisible load, mixed with other traffic without supervision. Individual axle loads and the gross vehicle weight may exceed the non-permit legislated limits. For the lane carrying the PA vehicle, the load effects shall be calculated from the more severe of
>
> (a) the permit vehicle alone in the lane with dynamic load allowance, or
>
> (b) 85% of the permit vehicle, plus a superimposed uniformly distributed load of 9, 8, 7 and 7 kN/m for highway classes A, B, C, and D, respectively, without dynamic load allowance for either Truck or uniformly distributed loads [3]."

Since a Normal Crossing defined by the MLC system has a DLA applied, the aforementioned case of a PA vehicle with DLA applied is applicable in MLC analysis. Furthermore, all military vehicles are assigned an MLC as their mass and load effects are known; therefore, it is appropriate to consider the PA case. Table 2-2 outlines the recommended live-load factors corresponding to their reliability index for normal traffic and all types of analysis (*CSA S6-14 CHBDC* Table 14.8) [3]. Table 2-3 outlines the recommended live-load factors corresponding to their reliability index for PA traffic (excerpt from *CSA S6-14 CHBDC* Table 14.10) [3]. While analysis could follow a Statically Determinate, Sophisticated, or Simplified approach, only the factors for the Simplified approach are presented as that method best correlates to typical applications of the MLC system. "Short Span" load factors apply to beams up to 6 m long for shear effects, to beams up to 10 m long for moment effects, and in floor beams where the tributary spans are up to 6 m long for shear and moment effects [3]. "Other Span" load factors are used in all other conditions [3].

Table 2-2. Live-Load Factors, α_L, for normal traffic, for all types of analysis, and for all spans [3].

Reliability Index, β	2.50	2.75	3.00	3.25	3.50	3.75	4.00
α_L	1.35	1.42	1.49	1.56	1.63	1.70	1.77

Table 2-3. Live-Load Factors, α_L, for PA traffic, Simplified Analysis [3].

Spans	Reliability Index, β						
	2.50	2.75	3.00	3.25	3.50	3.75	4.00
Short Spans	1.48	1.55	1.62	1.70	1.78	1.87	1.96
Other Spans	1.28	1.34	1.40	1.47	1.53	1.60	1.67

CSA S6-14 CHBDC outlines additional factors for DLA values to be multiplied by, for permit vehicle loads travelling at restricted speeds, of:

- 0.30 for a vehicle speed of 10km/h or less;
- 0.50 for a vehicle speed greater than 10 km/h and less than or equal to 25 km/h;
- 0.75 for a vehicle speed greater than 25 km/h and less than or equal to 40 km/h; and
- 1.00 for a vehicle speed greater than 40 km/h [3].

When bridges are assessed for permit vehicle loads using the mean load method, the bias coefficient (δ) and coefficient of variation (*v*) may be used in the absence of more reliable information [13]. The bias coefficient is a ratio of the mean and nominal effects, and the coefficient of variation is a ratio of standard deviation and mean [13]. The mean load method does not require the calculation of load or resistance factors, and statistical parameters are used instead. Values for the statistical parameters of DLA are shown in Table 2-4. While the mean load method is only appropriate in certain situations and as an alternate method, the statistical parameters could prove useful in comparison with test results in order to assess the validity of results.

Table 2-4. Statistical Parameters for Dynamic Load Allowance [13].

Span	δ	*v*
Short	0.67	0.60
Other – 1 lane loaded	0.60	0.80
Other – 2 or more lanes loaded	0.40	0.80

MacDonald (2014) provided an in-depth analysis in order suggest live-load factors for specific military vehicles based on a reliability index of β = 3.75, and recommends a uparmoured LAV III-ISC use 1.65 and a Leopard 2A4M tank use 1.38. A considerable amount of research would be required in order to recommend individual live-load factors for each military vehicle in Canada, not to mention all of NATO. Yet, there could be benefit to generating vehicle specific factors based on their calculated performance. Due to the variability in the live-load effects of military vehicles, MacDonald (2014) outlines a broader approach in which four different military vehicle categories are suggested. Recommended live-load factors are provided based on a reliability index of β = 3.75 for the four different military vehicle categories [10]:

- Wheeled-Transport (W-T): 1.77;
- Wheeled-Fighting (W-F): 1.48;
- Tracked-Transport (T-T): 1.77; and
- Tracked-Fighting (T-F): 1.33.

Further work is required to recommend appropriate live-load factors for lower levels of reliability indexes suitable for assessment and the increased risk of military operations.

Tracked fighting vehicles' (e.g. tanks) ability to cross bridges represents a critical aspect of military operations. As military vehicles increase in mass due to technological advancements and increases in armored protection, their mobility over existing infrastructure is reduced. The use of the same DLA for wheeled and tracked vehicles is likely inappropriate due to the perceived difference in the dynamic behavior of wheeled and tracked vehicles and may inappropriately limit the MLC capacity assigned to a bridge when supporting tank traffic. The lack of research into the differences between wheeled and tracked military vehicles requires examination. A bridge load test to validate these differences should be carried out.

2.4 Dynamic Load Allowance

2.4.1 Background

CSA S6-14 CHBDC defines DLA as "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 [3]." The phenomenon that occurs as vehicles cross bridges and generate a dynamic interaction is referred to differently in respective nations' design codes. A more in-depth examination of several countries design codes will be examined below, but it is important to note similar terms that are used in literature including: Dynamic Impact Factor (IM) [17], Dynamic Amplification Factor (DAF) [17] [18], or Dynamic Load Factor (DLF) [17]. In Canadian applications, DLA is applied to "all parts of the structure where force effects due to the gravity portion of moving loads may be present, including sidewalks, bearings, and substructures [13]."

The term DLA represents the values used in design code implementation. The use of the term DAF is often used in representing the amount of which static effects are increased by the bridge-vehicle interaction in practical field testing [18]. DAF is be represented by the term (1 + *DA*) in Equation 2-1 [18]:

$$R_{dyn} = R_{sta}(1 + DA) \qquad \text{2-1}$$

where R_{dyn} is the maximum dynamic response of the bridge, R_{sta} is the maximum static response of the bridge, and DA is the abbreviation for Dynamic Amplification. The term DAF is used in European code [17] which could lead to misinterpretation if the term DAF is used to represent the (1 + *DA*) term shown in Equation 2-1 for experimental results. Therefore (1 + *DA*) will be represented by Dynamic Effects Factor (DEF) and experimental results will be calculated using Equation 2-2:

$$DEF = \frac{R_{dyn}}{R_{sta}} = (1 + DA) \qquad 2\text{-}2$$

DLA values used in design codes would be representative of the experimental DA term. The relationship between all the similar terms can be expressed as:

$$DLA \approx IM \approx DA \approx DAF - 1 \approx DEF - 1 \approx DLF - 1 \qquad 2\text{-}3$$

with DA and DEF being the result of experimentation and DLA, IM, and DAF design code values.

Several factors can effect the dynamic interaction between bridges and the crossing vehicle loads including the vehicle type, the vehicle weight, the vehicle position with respect to a reference point, the surface condition, and multilane loading [19]. Bridge dynamic field testing is generally conducted by specific test vehicles; therefore this leads to values being generated that cannot accurately represent the conditions from normal traffic conditions [19]. In order to develop DLA values to be used in design codes, it is necessary to gather significant and variable traffic data over an extended period. Lighter vehicles tend to demonstrate a higher DEF in testing [20]. It was also noted that particularly on spans greater than 30 m, increased mass can result in a lower DEF [21]. Vehicle position with respect to instrumentation location is a factor that needs to be considered. Structural members that are out of the zone of influence will take small portions of the static load, yet their dynamic amplification can be large [19]. The effects of vehicle position will be further discussed in Section 2.4.3. The quality of the riding surface of both the bridge wear surface and the bridge approach can have significant influence on the dynamic magnification load effects on the bridge [19]. The condition of the approach has been shown to cause significant dynamic effects as an approach in poor condition can create large initial oscillations that may exceed design code values for DLA [17]. There is evidence that the number of lanes loaded can effect DEF values in testing [19]. The indication is that a single vehicle would generate a higher DEF than if many lanes were loaded. Single lane loading may overall represent more conservative DEF values, yet more accurately represent military loading.

The concept of understanding bridge dynamics has interested researchers dating back to 1849 [22], yet there is no consistency in its application that is further presented in Section 2.4.2. Understanding the dynamic phenomenon between vehicle loads and bridges continues to be a complex subject, with each bridge, site, and vehicle condition contributing their own unique parameters to the overall behaviour of the system. Throughout all studies, researchers tend to focus on wheeled loads, and relating the effects of these vehicles to design code provisions. There is no recent research for modern tracked vehicles, and specifically none for military tracked vehicles on non-military bridges. Analytical models have been developed yet there remains little consensus on which one to use. Due to the individual unique parameters of every situation, practical field testing remains the most viable solution to understand the dynamic effects generated by vehicle loads. It is recognized that field testing has generally focused on composite steel girder bridges as they are a predominant form of construction in North America, yet studies have been carried out on other bridge types [20].

Testing for dynamic effects of the vehicle bridge interaction is generally focussed on a smooth surface or testing bridges with “as-is” conditions. From a design and maintenance perspective, the bridge surface condition should be and should remain relatively smooth thus creating minimal dynamic effects as vehicles traverse the span; however, an object falling off a vehicle or the accumulation of snow/ice can create a sudden irregularity in the riding surface [19]. It is important to underline that the design loading for failure, for bridges with more than one lane, will correspond to the rare event of the bridge being overloaded by exceptionally heavy vehicles in multiple lanes [19]. As such, derivation of a DLA value should focus on smooth surfaces; however, the examination into the effects that surface irregularities cause to military traffic is important in developing multiple DLA values depending on the bridge condition.

2.4.2 Various Design Code Implementation

Dating back to the late 1920s, a simple expression was used to account for the complex interaction between bridges and their vehicular load and was calculated with the following equation [13]:

$$I = 15/(L + 38) \tag{2-4}$$

where *I* refers to the impact fraction (not to exceed 0.30), and *L* the span length in meters. The current version of the *CHBDC*, *S6-14*, provides more guidance for the application of a DLA, shown in Figure 2-5, and it is to be 0.50 for deck joints, 0.40 where only one axle of the design truck is used, 0.30 where any two axles of the design truck are used, and 0.25 when three axles of the design truck are used, shown in Figure 2-5 [3]. The aforementioned *CSA S6-14 CHBDC* DLA value are multiplied by 0.70 for wood components [3].

3.8.4.5.3 Components other than buried structures

For components other than buried structures, the dynamic load allowance shall be
(a) 0.50 for deck joints;
(b) 0.40 where only one axle of the CL-W Truck or the Special Truck(s) is used (except for deck joints);
(c) 0.30 where any two axles of the CL-W Truck, or axles nos. 1 to 3 of the CL-W Truck or the tandem or tridem of the Special Truck(s), are used; or
(d) 0.25 where three axles of the CL-W Truck, except for axles nos. 1 to 3, or more than three axles of the CL-W Truck, or more than one axle unit of the Special Truck(s), are used.

Subject to Approval, the dynamic load allowance given in Items (a) to (d) may be reduced by multiplying by 0.75 for a Special Truck travelling alone on the bridge under supervision, provided the speed of the Special Truck does not exceed 40 km/h when travelling on the bridge.

Note: *The axle numbers are shown in Figure 3.2.*

Figure 2-5. Excerpt from CSA S6-14 CHBDC on DLA [3]

Previous versions of the *Ontario Highway Bridge Design Code* (OHBDC 1983) used the first flexural frequency of a bridge in order to establish the relationship of which DLA value to use, shown in Figure 2-6 [13].

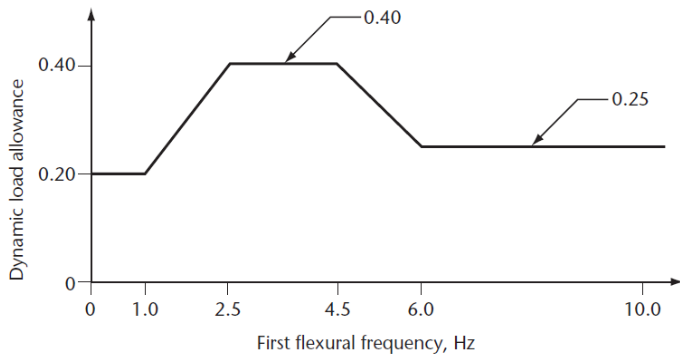

Figure 2-6. DLA/Frequency Relationship [13].

Deng et al. (2015) provided a comprehensive overview that reviews the various provisions of several countries design code values for DLA including the United States of America (USA), China, New Zealand, European Union (EU), Britain, and Japan. It is noted that each country vary their implementation of a DLA and that implementation of a DLA value has been simplified despite the effects being the result of many factors [17].

USA's AASHTO (1992) *Standard Specification for Highway Bridges* specifies the IM to be calculated as a function of bridge span length, *L*, in meters [23] which appears to be the same as Equation 2-4 with more decimal places due to metric/imperial conversions:

$$IM = \frac{15.24}{L + 38.10} \leq 0.3 \tag{2-5}$$

In the 2012 *AASHTO Load and Resistance Factor Design* (LRFD), DLA values were provided as shown in Table 2-5 [24]. DLA (previously IM) factors are no longer dependent on bridge span length.

Table 2-5. DLA Values in AASHTO 2012 [24].

Component	Limit State	DLA (%)
Deck Joint	All	75
All Other Components	Fatigue and fracture	15
	All other	33

China follows their *General Code for Design of Highway Bridges and Culverts*. The 1989 edition provides IM equations as a function of bridge span length, *L*, for the main structural members of concrete bridges (Equation 2-6) and main structural members of steel bridges (Equation 2-7) [25].

$$IM = \begin{bmatrix} 0.3 & L \leq 5\,m \\ 0.3 \times (1.125 - 0.025L) & 5\,m < L < 45\,m \\ 0 & L \geq 45\,m \end{bmatrix} \tag{2-6}$$

$$IM = \frac{15}{37.5 + L} \tag{2-7}$$

The 2004 edition was modified for the IM to be a function of the bridge fundamental frequency, *f*, shown in Equation 2-8 [26].

$$IM = \begin{bmatrix} 0.05 & f < 1.5\,Hz \\ 0.1767 lnf - 0.0157 & 1.5\,Hz \leq f \leq 14\,Hz \\ 0.45 & f > 14\,Hz \end{bmatrix} \tag{2-8}$$

New Zealand follows their 2013 New Zealand Transport Agency *Bridge Manual* that uses the DLF term. Moments in cantilevers and deck slabs, reaction, and shears use a DLF value of 1.30. Moments in simple and continuous spans is calculated as a function of the bridge span length, *L*, shown in Equation 2-9 [27].

$$DLF = \begin{bmatrix} 1.30 & L \leq 12\,m \\ 1 + \dfrac{15}{L + 38} & L > 12\,m \end{bmatrix} \tag{2-9}$$

The 2003 EU design code *Eurocode 1: Actions on Structures – Part 2: Traffic Loads on Bridges* uses the DAF term. The DAF is a function of bridge span length, *L*, and the number of traffic lanes govern which equation is used. Single lane bridges' DAF is calculated with Equation 2-10 and two-lane bridges' DAF is calculated with Equation 2-11 [28].

$$DAF = \begin{bmatrix} 1.7 & L \leq 5\,m \\ 1.85 - 0.03L & 5\,m < L < 15\,m \\ 1.4 & L \geq 15\,m \end{bmatrix} \tag{2-10}$$

$$DAF = \begin{bmatrix} 1.3 - \dfrac{0.4}{100}L & L \leq 50\,m \\ 1.1 & L > 50\,m \end{bmatrix} \tag{2-11}$$

The 2006 British Code follows BS 5400-2, *Steel, Concrete and Composite Bridges. Part 2: Specification for Loads* and uses an IM of 0.25 for both normal and abnormal traffic loads [29].

The New Zealand Design Code and the Eurocode are similar to the Japanese Design Code as their IM values are also a function of bridge span, *L*, as seen in Table 2-6 [30]. As shown in Table 2-6, truck loading is the same no matter the type of bridge, while the lane loading provision varies [30].

Table 2-6. Japanese IM Factors [30].

Bridge Type	Loading Type	IM
Steel	Truck and Lane	$20/(50 + L)$
RC	Truck	$20/(50 + L)$
	Lane	$7/(20 + L)$
Prestressed Concrete	Truck	$20/(50 + L)$
	Lane	$10/(25 + L)$

Evidently all countries implement the use of a form of a DLA; however, the parameters controlling implementation vary from span length, fundamental frequency, bridge type, or bridge component. Equation 2-4, from the 1920s, is the most prevalently used calculation for DLA; it is a function of bridge span length and there are similar implementations in 1992 AASHTO, 1989 Chinese Design Code, and 2013 New Zealand Design Code for spans greater than 12 m. The lack of consistency between national design codes for wheeled vehicles indicate that the dynamic effects generated by vehicles is still not fully understood nor in consensus between nations. More practical experimentation to increase the knowledge and data supporting DLA code provisions is required.

2.4.3 Testing Theory and Result Calculations

Early testing to understand the dynamic effects generated by moving vehicle loads across bridges often used the comparison of static and dynamic deflection measurements. Due to the use of both deflection and strain to measure the dynamic increase, the use of the term static or dynamic response is used in referring to the results obtained. DEF values obtained whether through deflection or strain measurements are considered equally valid; yet, it has been repeatedly demonstrated that DEF values computed from deflection measurements are always greater than when computed from strain measurements [19]. Due to technological advancements and the difficulties often encountered with deflection measurements, the use of strain gauges has become more commonplace when computing DEF values.

All DEF values are calculated using values obtained from a static response test. Five methods to calculate the static response are used:

1) test vehicle stationary over the instrumented location;
2) when the test vehicles crawls across the bridge at low speeds (5-15 km/h);
3) taking several measurements for different positions of the vehicle at rest;
4) use a low-pass digital filter to "smooth out" the dynamic frequencies in the signal; or
5) using finite-element modelling to compute the static displacement or strain from the given weight of a test vehicle [18].

The combination of a crawl and stationary test would prove to maximize the validity of stationary results, as the stationary positions may not generate the maximum static response depending on axle positioning and loading. Tracked vehicles centered over instrumentation should provide a clear static response due to the nature of their distributed loading. In analyzing the results of a crawl and a stationary test, it is important to examine the data in-order-to insure that the maximum values measured are not a result of braking to, or accelerating from, a stationary position.

Research has shown that the DEF values obtained in testing will be higher at a reference point away from the load than those from a reference point directly below the load [19]. When capturing the static load response of a bridge, the structural members directly under the applied load will carry an increased amount of load than those structural members not directly under the applied load. Yet those structural members located away from the applied load will result in higher DEF values due to their low static response. An expression was developed to account for the position of the applied load with respect to relevant structural members:

$$\alpha = \frac{D}{H + .5W} \tag{2-12}$$

where

$\alpha =$ relative position parameter with respect to a reference point
$D =$ distance in the transverse direction between the reference point and nearest line of wheels
$H =$ depth of bridge at instrumented cross-section
W = half-width of vehicle

If the value of α is less than 1.0, than the reference point is assumed to be relevant and lie within the zone of direct influence [31] as seen in Figure 2-7. The direct zone of influence is essentially anything contained within 45˚ of the test vehicle wheel path.

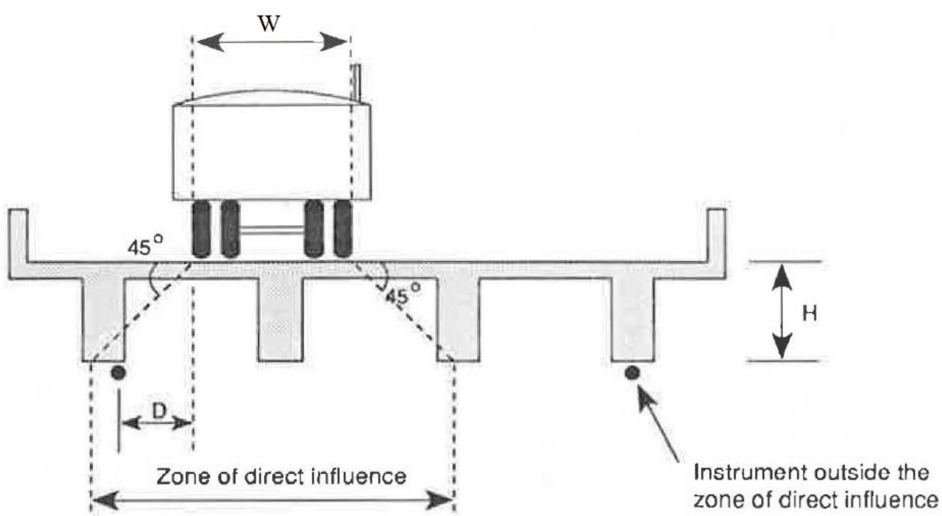

Figure 2-7. Zone of Direct Influence Diagram [31].

The zone of direct influence described above and seen in Figure 2-7 may be inappropriate for some bridge types due to variation in transverse load distribution. Therefore, it has been determined more appropriate and accurate to limit data from instrumentation at reference points that had the maximum static load response [19]. Furthermore, calculated DEF values can only be considered realistic if the extraneous data outside the zone of influence is excluded [19].

As previously indicated, instrumentation located at the same cross-section will provide different DEF values depending on vehicle location and which instrument values are used. A consideration in calculating the DEF is to divide the largest dynamic response by the largest static response, even if the data is from different locations from the instrumented cross-section. The values used should be from the same type of instrumentation (e.g. strain gauges data compared with strain gauge data, or displacement data compared with displacement data). This alternative approach will also prevent overestimation of DEF values [18]. Therefore, if there is multiple instrumentation located within the zone of influence, it is appropriate to divide the largest dynamic response by the largest static response resulting in a single DEF value per test, per instrumented location (e.g. negative and positive moment areas). It is unlikely that test vehicles will remain in the exact same line of travel between their static and dynamic response tests, therefore comparing only the largest values within the zone of influence assists in accounting for this as the absolute maximums are compared.

In a simple span bridge there will be no negative moment areas, therefore DEF values can only be calculated from the maximum positive moment area. However, continuous span bridge present an opportunity to examine DEF values obtained from the negative moment area as well.

During testing for DEF values there will be a natural scatter in the values obtained. Therefore, a method is required in order to establish DLA values for design or assessment. The maximum observed DEF value should not be used as a DLA value as it would likely prove to be overly conservative [19]. A method to derive DLA values exists that depends on the statistics of the amplification factor, the live-load factor, and the reliability index used seen in the following revised expression [21]:

$$DLA = \frac{(\overline{DEF} - 1)(1 + vs\beta)}{\alpha_L} \qquad \text{2-13}$$

where

$\overline{DEF}$ =	mean dynamic effects value
v =	coefficient of variation of the DEF, e.g. the ratio of standard deviation and mean minus one
s =	separation factor for dynamic loading, 0.57
β =	the safety index, typically 3.5 for highway bridges
α_L =	live-load factor

Differences exist in the dynamic behaviour of wheeled versus tracked military vehicles, and there is a lack of experimental studies for wheeled and tracked military traffic on spans greater than 15 m [10]. As discussed in Section 2.3, live-load factors for military traffic remain debatable. It would be prudent to examine calculated DLA values in order to examine the differences for different classes of military vehicles. Use of various safety indexes should also be explored in-line with the STANAG 2021 recommended β value of 3.3 [4], lower values of β suitable for military operations, and the various provisions outlined in *CSA S6-14 CHBDC* in order to examine the relationship between Reliability Index, live-load factors, and recommended DLA values.

2.5 Dynamic Testing With Surface Irregularities

As detailed in Section 2.4.1, testing for dynamic effects of the vehicle bridge interaction in order to generate code provisions for DLA is generally focussed on a smooth surface or testing bridges with "as-is" conditions. The approach of placing a wooden plank or other obstacle on the travel path to increase the dynamic effects observed is recognized by researchers, but not used in all dynamic testing. As noted, an object falling off a vehicle or the accumulation of snow/ice can create a sudden irregularity in the riding surface [19]. While during peacetime military exercises, one would not expect there to be deliberate obstacles on the riding surface, it is in the realm of possible during military operations. Therefore, the testing of dynamic amplification over obstacles carries merit in testing with military vehicles.

The use of artificial surface irregularities in order to examine the dynamic interaction between bridges and vehicles has been seldom carried out; even with the consensus among researchers that surface irregularities will cause an increase in dynamic effects. Bridge surface conditions are assumed to be as smooth as possible in design, and proper maintenance is carried out to insure the roadway surface does remain as smooth as possible. A study was carried out during 1976 in Britain to examine the dynamic effects generated by a test vehicle travelling over planks, in order to compare it to the maximum permissible surface irregularity height. Data from testing over obstacles of 20 mm, 30 mm, and 40 mm was extrapolated and determined that an obstacle of 9 mm would generate a peak DEF of 1.4 [32]. These results assist in demonstrating the significant increase in dynamic effects that can occur from relatively small surface irregularities.

Interestingly, all bridges in Latvia undergo dynamic load tests prior to bridge commissioning that include vehicles travelling over smooth and uneven roadway conditions [33]. The uneven pavement tests are representative of damaged pavement or ice bumps and are created by timber planks approximately 50 mm tall and 100 mm wide placed on the bridge wearing surface. Planks are generally 3.0 m to 3.5 m apart over 2/3 of the span length [33]. These represent fairly significant obstacles considering the height and repeated pattern. A study of results from bridge load tests from 1991 until 2012 notes, as one might expect, scattering in DEF results and that bridge type appears to be more controlling in the dynamic responses observed vice span length as seen in most design code provisions [33]. The use of timber planks as obstacles in a repeated pattern is a noteworthy method for creating artificial surface irregularities in dynamic testing.

Situations may arise in military operations where bridges have been damaged or deliberate obstacles have been place across them. It is necessary to clearly understand the magnitude of dynamic load increase to structures generated by surface irregularities (be they surface roughness, combat damage, or deliberate obstacles) to avoid the risk of overloading should vehicles have to traverse any irregularity.

2.6 Braking Effects

Vehicles will apply a transient longitudinal force to the top of the bridge when braking, which causes a longitudinal motion in the superstructure that is restrained by support reactions [13]. Many factors must be considered in the analysis of braking force including the coefficient of friction between the tires and the wearing surface, and dynamic characteristics of: (1) a bridge's superstructure; (2) the vehicle; and (3) bridge bearing and piers. *CSA S6-14 CHBDC* has simplified the braking effects for ease of design and outlines specifications for how the braking force is to be considered:

> "Braking force shall be considered only at the ultimate limit states.
> Braking force shall be an equivalent static force of 180 kN plus 10% of the uniform distributed load portion of the lane load from one design lane, irrespective of the number of design lanes, but not greater than 700 kN in total.
> The braking force shall be applied at the deck surface" [3].

It is also noted that "Under extreme conditions, the dynamic braking force due to an axle may be as much as 80% of the axle-load" [13].

The simplified application presented in Canadian design code only applies braking as a transient longitudinal force. However, when a vehicle brakes, there is a shift in the load between the axles, changing the vertical forces, and resulting in increased flexural forces. *CSA S6-14 CHBDC* does acknowledge that braking effects will cause dynamic increases, as it is assumed that the sudden application of brakes is unlikely in permit situations that have applied a reduction in DLA [13]; however, the DLA provisions give no indication that they include the dynamic effects as a result of braking. It is further specified that DLA "is not required for centrifugal, braking, collision or pedestrian loads" and that braking is considered a rare event likely to not occur to overloaded vehicles [13].

The National Cooperative Highway Research Program (NCHRP) conducted research into determining the maximum braking force as a fraction of the vehicle weight and determined it to be approximately 25% [13]. The NCHRP value was determined using a stopping distance of 122 m from a speed of 88.5 km/h [13]. Yet research has shown that a shorter braking duration can significantly increase the braking effects [34]. Research into the interaction between vehicle braking and the dynamic increase appears limited. A study where vehicles come to a full stop as rapidly as possible would prove valuable for determining the dynamic effects that are generated from rapid braking, and to examine how they relate to the dynamic allowances from design code provisions.

2.7 Summary

The intent of this literature review was to provide the reader with sufficient background information on the various topics of this research project. The practice of calculating dynamic effects is generally the same throughout all examined literature; however, there are perspectives on which data is critical in calculations depending on a specific site setup. It is also evident that there are varying methods for the application of DLA in various countries. There remains little research into the load effects of military vehicles, and a clear dearth of research on the different dynamic effects generated by wheeled or tracked military vehicles. Research into the dynamic effects generated by surface irregularities is limited as design codes assume a smooth trafficked surface, and there is limited guidance and evidence on the effects that surface irregularities may cause. Literature appears to support the notion that vehicle braking will generate a dynamic effect, yet it is not quantified and is applied longitudinally to the bridge. As such, there are several areas where there appears to be a lack of research that this project hopes to address. Each manuscript contains an overview of specific literature in their introduction sections as they have the capacity to be standalone documents.

3. MANUSCRIPT #1: "DYNAMIC LOAD EFFECTS OF WHEELED AND TRACKED MILITARY VEHICLES ON A STEEL GIRDER COMPOSITE BRIDGE"

3.1 Abstract

The perceived and observed differences in the dynamic behaviour between wheeled and tracked military vehicles should be accounted for in the application of appropriate dynamic load effect values for bridge design and assessment. No current bridge design or assessment code provides guidance on methods to differentiate between the dynamic loading effects of wheeled and tracked vehicles. Depending on the code or guideline being applied for analysis, these dynamic loading effect values may be represented and referred to as Dynamic Load Allowance (DLA), Dynamic Amplification Factor (DAF), or Dynamic Impact Factors (IM). The North Atlantic Treaty Organization (NATO) utilizes a Military Load Classification (MLC) System to compare the load effects of vehicles to the capacity of bridges in order to determine the feasibility of crossing. Civilian codes and the MLC System use the same dynamic loading effects values, for both wheeled and tracked vehicles, which can significantly impact and limit the mobility of tracked vehicles, particularly main battle tanks, on military operations.

Bridge load testing was carried out to compare the dynamic loading effects between three wheeled military vehicles and Canada's main battle tank, the Leopard 2. The dynamic effects were then used to calculate suggested DLA values for individual vehicles and per vehicle category (wheeled and tracked) using various combinations of recommended design code reliability indexes and live-load factors. Results indicate that it may be appropriate to reduce the DLA used for military tracked vehicles by one-third of that used for military wheeled vehicle analysis. A review of several nations' DLA values was carried out, and application of a reduced DLA for tracked vehicles could result in an increase to predicted bridge capacity of 5% to 13% for tracked vehicles.

3.2 Introduction

3.2.1 Military Load Classification System

Route identification and selection is a critical task completed by military commanders when planning missions and operations. A key component of route selection is knowing which bridges can support the required military vehicle traffic. Furthermore, when armed forces conduct exercises or operations, at home or abroad, military vehicles will undoubtedly be required to cross civilian bridges; it is essential that loading effects and bridge capacities are understood and quantified. The North Atlantic Treaty Organization (NATO) utilizes the Military Load Classification (MLC) System outlined in NATO Standardization Agreement (STANAG) 2021 Edition-8 to calculate the MLC of bridges and vehicles. The aim of STANAG 2021 is to provide NATO forces with a standard method for computing the MLC representing the load capacities of all bridges, military ferries and rafts, and the loading effects of military vehicles [1]. It is important to note that the MLC value is only a number representing the loading effects of vehicles and does not directly indicate the mass of the vehicle [1], or a tonnage capacity value for a bridge.

The MLC system uses 32 hypothetical vehicles (16 wheeled and 16 tracked), that are based on representative NATO nations' vehicles, to create unfactored graphs of their unit bending moments and shear effects on bridge spans ranging from 1 m to 100 m [1]. Different curves are derived for wheeled and tracked vehicles. A representative set of curves for the unit bending moment of theoretical wheeled vehicles is shown in Figure 3-1, and for the unit bending moment of theoretical tracked vehicles is shown in Figure 3-2.

MLC values for vehicles are determined by calculating the governing shear and moment loading effects of that vehicle, on spans ranging from 1 m to 100 m, and these results are then compared to the loading effects generated by theoretical vehicles. Vehicles are assumed to be travelling in convoys with 30.5 m spacing (100 feet) between the trailing and leading axles of consecutive vehicles [1]; therefore, the effects of multiple vehicles are considered in spans greater than 30.5 m. The maximum shear and moment values are then compared to the theoretical vehicle load effects; and whichever span length yields the highest MLC value, will govern the MLC value for that vehicle. The benefit of analyzing vehicles through this method, vice using its number of axles and Gross Vehicle Weight (GVW), is that it permits each vehicle to be rated on the nominal maximum effects that it generates on a simple span [2]. MLC values should be calculated when vehicles are fully loaded with fuel, ammunition, personnel, kit, load, etc. This may in turn limit the mobility of some vehicles, as they may not often be operating at a fully laden condition.

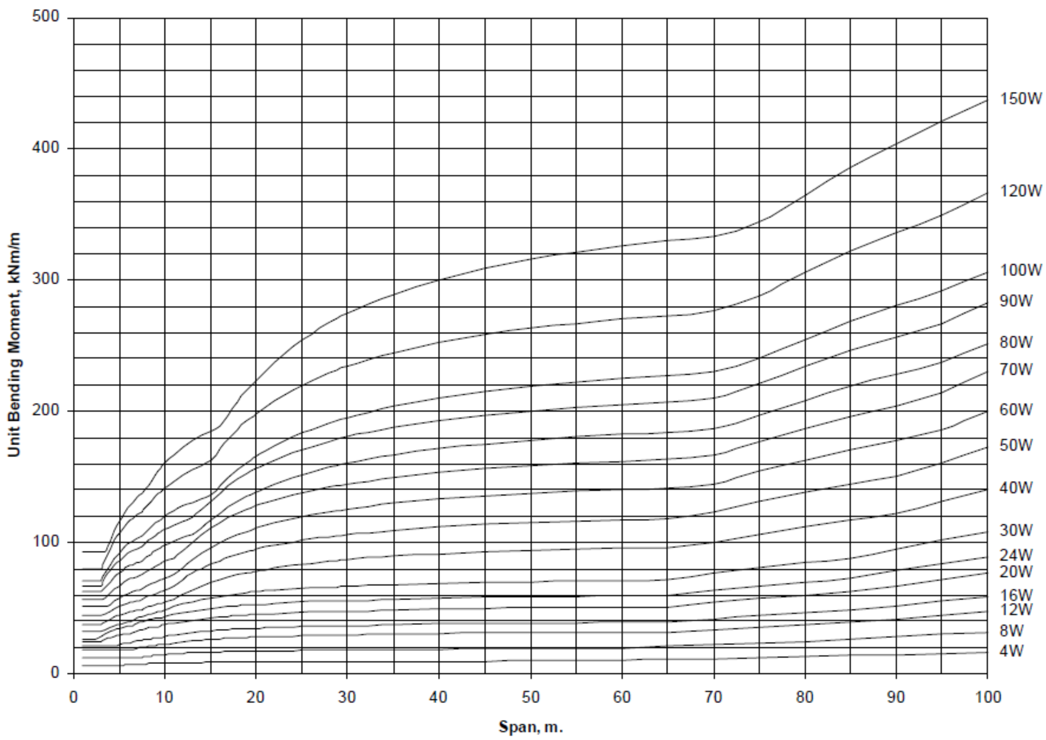

Figure 3-1. Unit Bending Moments due to Theoretical Wheeled Vehicles - Spans from 1 m to 100 m [3].

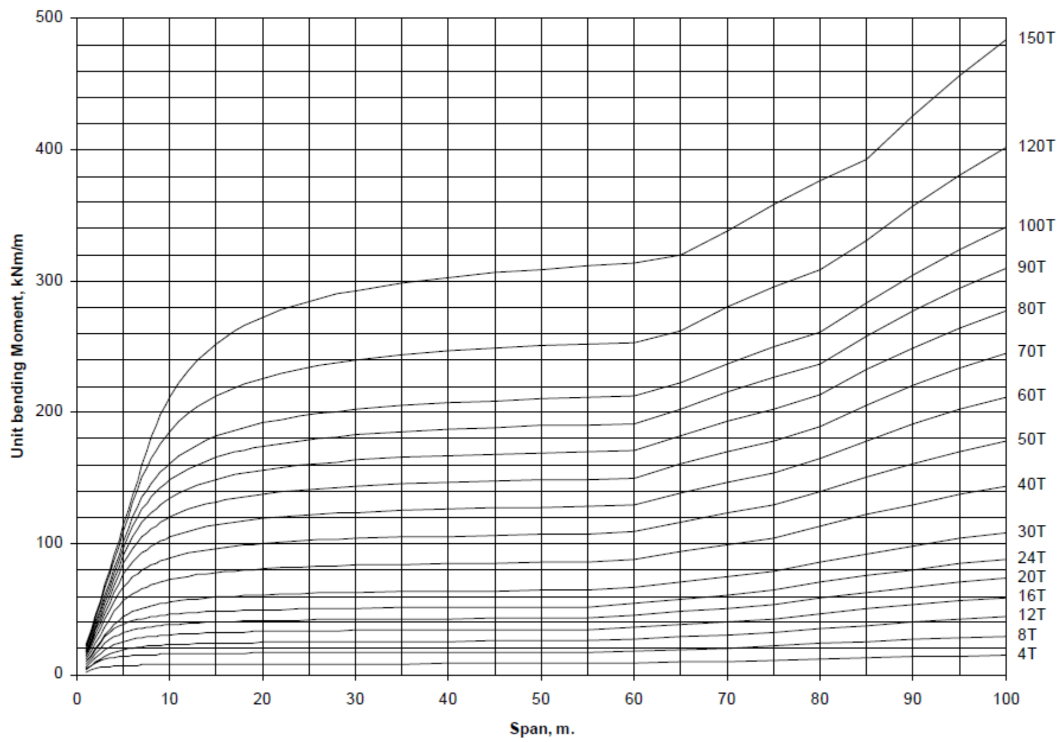

Figure 3-2. Unit Bending Moments due to Theoretical Tracked Vehicles - Spans from 1 m to 100 m [3].

To assign an MLC capacity rating to a bridge, it is necessary to determine the allowable shear and moment capacity of that structure. Once the allowable shear and moment values are determined, their values are plotted using the bridge span on the graphs similar to those seen in Figure 3-1 and Figure 3-2. Because the load effects of wheeled and tracked military vehicles are unique and different from normal civilian traffic [2], it is necessary to appropriately account for this in determining MLC capacities of bridges.

The MLC system described in STANAG 2021 also establishes three crossing conditions of Normal, Caution, and Risk Crossing. Normal Crossing of a bridge permits unrestricted use of civilian bridges and normal accepted restrictions for any military bridges [1]. This research project was carried out in order to validate the difference in the dynamic loading effect behaviour between wheeled and tracked military vehicles, subject to Normal Crossing conditions.

3.2.2 Load Factors

The live-load capacity of a bridge can be estimated by determining the resistance of the structure and then subtracting the effects of the dead load. To safely assess a bridge's capacity, factors are applied to decrease the resistance of the bridge and increase the effects of the loads. These adjustments account for a variety of issues including the variability of material properties, quality of construction, and the actual effects of traffic, and are typically determined using reliability-based methods at the ultimate limit state. The live-load factor specifically accounts for unknown overloads during a bridge's lifetime and uncertainties in the load analysis [4]. Although STANAG 2021 is established to provide NATO forces with a standard method for computing

MLCs, no specific method is imposed and each country is permitted to use their own procedures and safety factors [1] [5]. Therefore, STANAG 2021 has moved towards the development of a safety concept while allowing individual countries to follow their civilian codes while establishing national standards [1]. STANAG 2021 has suggested the use of a Safety Factor, or Reliability Index, β, of 3.3 for Normal Crossings in bridge classification [1]. This allows NATO nations to apply their national standards from their respective codes, for example the Canadian S6-14 Canadian Highway Bridge Design Code (CHBDC), the American Association of State Highway and Transportation Officials (AASHTO) Manual for Bridge Evaluation, or the Eurocode. Within these standards, there is a dearth of guidance on assessing the load effects generated by tracked vehicles, be they civilian or military.

The use of Reliability Indexes, β, is prevalent in many nations' codes and is the recommended basis for all current Limit State Design (LSD) codes [4]. The Reliability Index is selected in order to provide an expected structural behaviour level of safety, which is inversely related to its notional probability of failure. As per *CSA S6-14 CHBDC*, new structures are required to achieve a target β value of 3.75 for a 75 year design life, while bridge inspection carries different suggested β values depending on the level of inspection and details of the structural system of the bridge [6].

The *CSA S6-14 CHBDC* used LSD philosophy in applying load and resistance factors when designing structures [6]. A structure, e.g. a bridge, should be designed so that when at the ultimate limit state (ULS), the factored resistance exceeds the total factored load effect [6]. All structural components are to comply with the ULS, the serviceability limit state (SLS), and the fatigue limit state (FLS) provisions outlined in the *CSA S6-14 CHBDC* [6]. As such, when assessing the capacity of existing infrastructure, many bridges' assessment indicate a reduced performance with modern design loading [7] as current bridge codes are primarily concerned with the design of new structures.

The *CSA S6-14 CHBDC* provides load combinations requiring consideration when assessing the FLS, SLS, or ULS of a structure. The load combinations fit into three categories of permanent loads, transitory loads, and exceptional loads [6]. Included in the transitory loads category are live-load factors, which include the DLA when applicable [6]. *CSA S6-14 CHBDC* establishes a DLA of a 0.25 for most loading situations and a live-load factor based on the reliability index, β [6]. STANAG 2021 suggests a live-load factor of 1.35 in MLC for both wheeled and tracked vehicles under normal crossing conditions; however, there is no suggested DLA values and it is recommended to follow individual country's standard practice depending on vehicle type and expected use of the bridge [1]. Canadian and United States military doctrine suggest a DLA of 0.15 for MLC application [3] [8].

CSA S6-14 CHBDC makes use of a Normal Traffic category and four classes of permit vehicles. One permit class, referred to as Annual or Project (PA), outlines a case where a vehicle of known weight is calculated alone in the lane with specified reliability index and live-load factors different than those of the Normal Traffic. A Normal Crossing defined by the MLC system has a DLA applied, and all military vehicles are assigned an MLC as their mass and load effects are known; therefore, it is appropriate to consider the PA values for reliability index and live-load factors. Table 3-1 outlines the recommended live-load factors corresponding to their reliability index for normal traffic and all types of analysis (*CSA S6-14 CHBDC* Table 14.8) [6]. Table 3-2 outlines the recommended live-load factors corresponding to their reliability index for PA traffic (excerpt from *CSA S6-14 CHBDC* Table 14.10) [6]. While analysis could follow a statically determinate, sophisticated, or simplified approach, only the factors for the simplified approach are presented as that method best correlates to typical applications of the MLC system.

Table 3-1. Live-Load Factors, α_L, for normal traffic, for all types of analysis, and for all spans [6].

Reliability Index, β	2.50	2.75	3.00	3.25	3.50	3.75	4.00
α_L	1.35	1.42	1.49	1.56	1.63	1.70	1.77

Table 3-2. Live-Load Factors, α_L, for PA traffic, Simplified Analysis [6].

Spans	**Reliability Index, β**						
	2.50	*2.75*	*3.00*	*3.25*	*3.50*	*3.75*	*4.00*
Short Spans	1.48	1.55	1.62	1.70	1.78	1.87	1.96
Other Spans	1.28	1.34	1.40	1.47	1.53	1.60	1.67

Note: Shorts Span - applies to beams up to 6 m long for shear effects, to beams up to 10 m long for moment effects, and in floor beams where the tributary spans are up to 6 m long for shear and moment effects.
Other Span - used in all other conditions

When bridges are assessed for permit vehicle loads using the mean load method, the bias coefficient (δ) and coefficient of variation (v) may be used in the absence of more reliable information [4]. The bias coefficient is a ratio of the mean and nominal effects, and the coefficient of variation is a ratio of standard deviation and mean [4]. The mean load method does not require the calculation of load or resistance factors, and statistical parameters are used instead. While the mean load method is only appropriate in certain situations and as an alternate method, the statistical parameters are useful in comparison with test results in order to assess the validity of results. A bias coefficient of 0.60 and a coefficient of variation of 0.80 would be applicable for assessment of this bridge should the mean load method be used [4].

Similar to the distinct behaviour that may be evident in the dynamic load effects of tracked and wheeled vehicles, based on observed probabilistic GVW of military vehicles, it is recommended for bridge analysis that live-load factors also be differentiated between military transport and military tracked vehicles [9]. MacDonald (2014) provided an in-depth analysis in order to suggest live-load factors for specific military vehicles based on a reliability index of β = 3.75, and recommends an up-armoured LAV III-Infantry Section Carrier (ISC) use 1.65, and a Leopard 2A4M tank use 1.38. A considerable amount of research would be required in order to recommend individual live-load factors for each military vehicle in Canada, not to mention all of NATO. Yet, there could be benefit to generating vehicle specific factors based on their calculated performance. Due to the variability in the live-load effects of military vehicles, MacDonald (2014) outlines a broader approach in which four different military vehicle categories are suggested. Recommended live-load factors are provided based on a reliability index of 3.75 for the four different military vehicle categories [2]:

- Wheeled-Transport (W-T): 1.77;
- Wheeled-Fighting (W-F): 1.48;
- Tracked-Transport (T-T): 1.77; and
- Tracked-Fighting (T-F): 1.33.

It may be noted that a reliability index of 3.75, or even 3.3, may be conservative for many military operations, and that much lower live-load factors may be appropriate for those operational conditions. Considering the inherent risk of military operations, it may be appropriate to accept more risk and to consider a lower value of reliability index when developing live-load factors [10]. Nevertheless, as discussed above, it is apparent that based on the characteristics of these vehicles, it is appropriate to apply significantly different live-load factors between transport and fighting vehicles.

Tracked fighting vehicles' (e.g. tanks) ability to cross bridges represents a critical aspect of military operations. As military vehicles increase in mass due to technological advancements and increases in armored protection, their mobility over existing infrastructure is reduced. The use of the same DLA for wheeled and tracked vehicles is likely inappropriate due to the perceived difference in the dynamic behavior of wheeled and tracked vehicles and may inappropriately limit the MLC capacity assigned to a bridge when supporting tank traffic.

3.2.3 Dynamic Load Allowance

3.2.3.1 Background

CSA S6-14 CHBDC defines DLA as "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 [6]." The phenomenon that occurs as vehicles cross bridges and generate a dynamic interaction is referred to differently in respective nations' design codes. A more in-depth examination of several countries design codes will be examined below, but it is important to note similar terms that are used in literature including: Dynamic Impact Factor (IM) [11], Dynamic Amplification Factor (DAF) [11] [12], or Dynamic Load Factor (DLF) [11]. In Canadian applications, DLA is applied to "all parts of the structure where force effects due to the gravity portion of moving loads may be present, including sidewalks, bearings, and substructures [4]."

The term DLA represents the values used in design code implementation. The use of the term DAF is often used in representing the amount by which static effects are increased by the vehicle bridge interaction in practical field testing [12]. DAF is be represented by the term (1 + *DA*) in Equation 3-1 [12]:

$$R_{dyn} = R_{sta}(1 + DA) \qquad \text{3-1}$$

where R_{dyn} is the maximum dynamic response of the bridge, R_{sta} is the maximum static response of the bridge, and DA is the abbreviation for Dynamic Amplification. The term DAF is used in European code [11] which could lead to misinterpretation if the term DAF is used to represent the (1 + *DA*) term shown in Equation 3-1 for experimental results. Therefore (1 + *DA*), representative of experimental results, will be denoted by Dynamic Effects Factor (DEF) and calculated using Equation 3-3:

$$DEF = \frac{R_{dyn}}{R_{sta}} = (1 + DA) \qquad 3\text{-}2$$

DLA values used in design codes would be representative of the experimental DA term. The relationship between all the similar terms can be expressed as:

$$DLA \approx IM \approx DA \approx DAF - 1 \approx DEF - 1 \approx DLF - 1 \qquad 3\text{-}3$$

with DA and DEF being the result of experimentation and DLA, IM, DAF, and DLF design code values.

Several factors can effect the dynamic interaction between bridges and the crossing vehicle loads including the vehicle type, the vehicle weight, the vehicle position with respect to a reference point, the surface condition, and multilane loading [13]. Bridge dynamic field testing is generally conducted by specific test vehicles; therefore, this leads to values being generated that cannot accurately represent the conditions from normal traffic conditions [13]. In order to develop DLA values to be used in design code, it is necessary to gather significant and variable traffic data over an extended period. Lighter vehicles tend to demonstrate a higher DEF in testing [14]. It was also noted that, particularly on spans greater than 30 m, increased mass can result in a lower DEF [15]. The quality of the riding surface of both the bridge wear surface and the bridge approach can have significant influence on the dynamic magnification load effects on the bridge [13]. The condition of the approach has been shown to cause significant dynamic effects as an approach in poor condition can create large initial oscillations that may exceed design code values for DLA [11]. Additionally, there is evidence that the number of lanes loaded can affect DEF values in testing [13]. The indication is that a single vehicle would generate a higher DEF than if many lanes were loaded. Testing with single lane loading may, overall, represent more conservative DEF values, yet more accurately represent military loading situations.

Understanding the dynamic phenomenon between vehicle loads and bridges continues to be a complex subject, with each bridge, site, and vehicle condition contributing their own unique parameters to the overall behaviour of the system. Throughout all studies, researchers focus on wheeled loads, and relate the behaviour of wheeled vehicles to design code provisions. There is no recent research for modern tracked vehicles, and specifically none for military tracked vehicles when applied to typical highway bridges. While analytical models have been developed, there remains little consensus on which one to use. Due to the individual unique parameters of every situation, practical field-testing remains the most viable solution to understand the dynamic effects generated by vehicle loads.

Testing for dynamic effects of the vehicle bridge interaction is generally focussed on a smooth surface or testing bridges with "as-is" conditions. From a design and maintenance perspective, the bridge surface condition should be and should remain relatively smooth, thus creating minimal dynamic effects as vehicles traverse the span; however, an object falling off a vehicle or the accumulation of snow/ice can create a sudden irregularity in the riding surface [13]. It is important to note that the design loading for failure, for bridges with more than one lane, will correspond to the rare event of the bridge being overloaded by exceptionally heavy vehicles in multiple lanes [13]. As such, derivation of a DLA value should focus on smooth surfaces.

3.2.3.2 Testing Theory and Result Calculations

Early testing to understand the dynamic effects generated by moving vehicle loads across bridges often used the comparison of static and dynamic deflection measurements. DEF values obtained whether through deflection or strain measurements are considered equally valid; yet, it has been repeatedly demonstrated that DEF values computed from deflection measurements are always greater than when computed from strain measurements [13]. Due to the use of both deflection and strain to measure the dynamic increase, the use of the term static or dynamic response is used in referring to the results obtained. Due to technological advancements and the difficulties often encountered with deflection measurements, the use of strain gauges has become more commonplace when computing DEF values.

All DEF values are calculated using values obtained from a static response test. Five methods to calculate the static response are used:

1. A test vehicle remains stationary over the instrumented location;

2. A test vehicle crawls across the bridge at low speeds (5-15 km/h);

3. The recording of several measurements are for different positions of the vehicle at rest;

4. The use of a low-pass digital filter to “smooth out” the dynamic frequencies in the signal of a moving load test; or

5. The application of finite-element modelling to compute the static displacement or strain from the given weight of a test vehicle [12].

The combination of a crawl and stationary test would reinforce the validity of stationary results, as the stationary positions may not generate the maximum static response depending on axle positioning and loading. Tracked vehicles centered over instrumentation should provide a clear static response due to the nature of their distributed loading. In analyzing the results from the crawl and a stationary test, it was important to examine the data in-order-to insure that the maximum values measured were not a result of braking to, or accelerating from, a stationary position.

Research has shown that the DEF values obtained in testing will be higher at a reference point away from the load than those from a reference point directly below the load [13]. When capturing the static load response of a bridge, the structural members directly under the applied load will carry an increased amount of load than those structural members not directly under the applied load. Yet those structural members located away from the applied load will result in higher DEF values due to their low static response. An expression was developed to account for the position of the applied load with respect to relevant structural members:

$$\alpha = \frac{D}{H + .5W} \qquad 3\text{-}4$$

where

α = relative position parameter with respect to a reference point
D = distance in the transverse direction between the reference point and nearest line of wheels
H = depth of bridge at instrumented cross-section
W = half-width of vehicle

If the value of α is less than 1.0, than the reference point is assumed to be relevant and lie within the zone of direct influence [16]. The direct zone of influence is anything contained within 45˚ of the test vehicle wheel path. The zone of direct influence may be inappropriate for all bridge types due to variation in transverse load distribution. Therefore, it has been determined more appropriate and accurate to limit data from instrumentation at reference points that had the maximum static load response [13]. Furthermore, calculated DEF values can only be considered realistic if the extraneous data outside the zone of influence is excluded [13].

As previously indicated, instrumentation located at the same cross-section will provide different DEF values depending on vehicle location and which instrument values are used. A consideration in calculating the DEF is to divide the largest dynamic response by the largest static response, even if the data is from different locations from the instrumented cross-section. The values used should be from the same type of instrumentation (e.g. strain gauges data compared with strain gauge data, or displacement data compared with displacement data). This alternative approach will also prevent overestimation of DEF values [12]. Therefore, if there are multiple instruments located within the zone of influence, it is appropriate to divide the largest dynamic response by the largest static response resulting in a single DEF value per test, per instrumented location (e.g. negative and positive moment areas). It is unlikely that test vehicles will remain in the exact same line of travel between their static and dynamic response tests, therefore comparing only the largest values within the zone of influence assists in accounting for this as the absolute maximums are compared. This revised method of dividing the largest dynamic response by the largest static response of the instrumentation within the zone of influence was used in calculating results for this research project. As the tested structure was a continuous span bridge, it presented an opportunity to calculated DEF values from the positive and negative moment areas.

During testing for DEF values there will be a natural scatter in the values obtained. Therefore, a method is required in order to establish DLA values for design or assessment, and to correlate test values to obtained results. The maximum observed DEF value should not be used as a DLA value as it would likely prove to be overly conservative [13]. A method to derive DLA values exists that depends on the statistics of the amplification factor, the live-load factor, and the reliability index used seen in the following revised expression [15]:

$$DLA = \frac{(\overline{DEF} - 1)(1 + vs\beta)}{\alpha_L} \qquad 3\text{-}5$$

where

$\overline{DEF}$ =	mean dynamic effects value
v =	coefficient of variation of the DEF
s =	separation factor for dynamic loading, 0.57
β =	the reliability index
α_L =	live-load factor

Differences exist in the dynamic behaviour of wheeled versus tracked military vehicles, and there is a lack of experimental studies for wheeled and tracked military traffic on spans greater than 15 m [2]. As discussed in Section 3.2.2, live-load factors for military traffic remain debatable. Different classes of military vehicles were used in this research in order to further validate this difference in vehicle behaviour. Various safety indexes were also used in order to examine the relationship between reliability index, live-load factors, and recommended DLA values for different traffic categories.

3.3 Research Objectives and Scope

This research paper is part of a larger research project that is also investigating the dynamic effects of military traffic over obstacles, the braking effects of military traffic, load distribution in a compositely-acting bridge, and further analysis on the effects of military traffic. The results presented herein are a result of testing on one bridge with four different military vehicles varying from 26 to 61 tonnes (one tracked and three wheeled), in order to examine the differences in the dynamic effects between wheeled and tracked vehicles. The main objective in validating the different dynamic behaviour was to develop a suggested reduced DLA value for tracked vehicles.

The static response of the bridge was obtained for all test vehicles along the centerline and edge position of the bridge. Testing was conducted in three categories: smooth surface passes, passes with obstacles across the bridge deck, and passes incorporating braking. The results discussed in this paper are a result of the smooth surface tests. Vehicles would conduct each test at varying speeds along the centerline and one edge position of the bridge. Varying speeds and positions generated significant data for the computation of results. DEF values were computed for each smooth surface test and at the positive and the negative moment locations of the bridge. DLA values were then calculated from various combinations of reliability index and live-load factors. The calculated DLA values for wheeled and tracked vehicles permit the examination and comparison of the differences in the dynamic behaviour of both vehicle types.

3.4 Experimental Program

3.4.1 Vehicles

Four different military vehicles were used for the testing. The tracked fighting vehicle used was Canada's main battle tank (MBT), the Leopard 2, shown in Figure 3-3. The wheeled vehicles used in testing were: a Heavy Logistics Vehicle Wheeled (HLVW) with a low-bed (Figure 3-4), an Engineer Light Armoured Vehicle (E-LAV) (Figure 3-5), and an Expedient Route Opening Capability (EROC) vehicle know as the Cougar (Figure 3-6). Detailed loading and schematics of the used vehicles are provided in Appendix F.

Figure 3-3. Canada's MBT, the Leopard 2A6M.

The Leopard 2 is an MLC 70 tracked vehicle, and weighs 63.2 tonnes when fully loaded. The tracks are 0.64 m wide with a ground contact length of 5.0 m. The Leopard 2 was not fully loaded with fuel or ammunition at the time of testing and therefore weighed only 60.6 tonnes.

Figure 3-4. Heavy Logistics Vehicle Wheeled (HLVW) with a Low-Bed.

The HLVW had a low-bed trailer attached to it that was loaded with a 27 tonne Zettelmeyer front-end loader. The total vehicle length was 18.7 m with a total vehicle width of 2.8 m, and total mass of 50.4 tonnes dispersed over the five axles, representative of an MLC 49 wheeled vehicle during testing.

Figure 3-5. Engineer Light Armoured Vehicle (E-LAV).

The E-LAV has a wheelbase of 4.2 m, with 2.8 m standard wheel track, and 3.4 m total width due to the various equipment attached to the sides of it. The maximum weight is 27.5 tonnes, representative of an MLC 30 wheeled vehicle.

Figure 3-6. Cougar.

The Cougar has a wheelbase of 4.9 m, with a 2.6 m standard wheel track, and 2.7 m total width. The maximum weight is 26.1 tonnes, representative of an MLC 29 wheeled vehicle.

3.4.2 Test Set-Up

3.4.2.1 Site and Bridge Information

The location for testing was the Mountbatten Bridge (Figure 3-7), located at 4th Canadian Support Base (CDSB) Petawawa in Ontario, Canada. The Royal Engineers of the British Army constructed the Mountbatten Bridge in 1977 and 1978. The bridge is a continuous structure with two spans of 29.6 m and 32.9 m, with four steel stringers acting compositely with a concrete deck. The bridge is 7.5 m wide overall with a 6.0 m roadway width. The cross section of a single girder, and the entire bridge are shown in Figure 3-8 and Figure 3-9 respectively.

Figure 3-7. Elevation View of the Mountbatten Bridge in Winter.

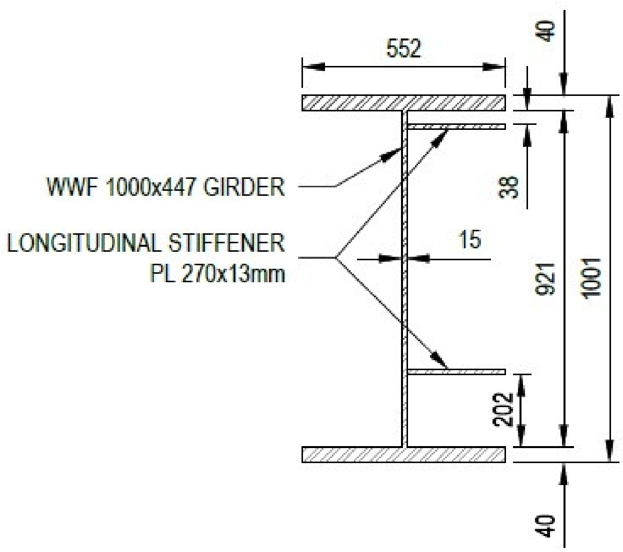

Figure 3-8. WWF 1000x447 Girder Properties (all dimensions in mm). Longitudinal Stiffeners are located in the negative moment location.

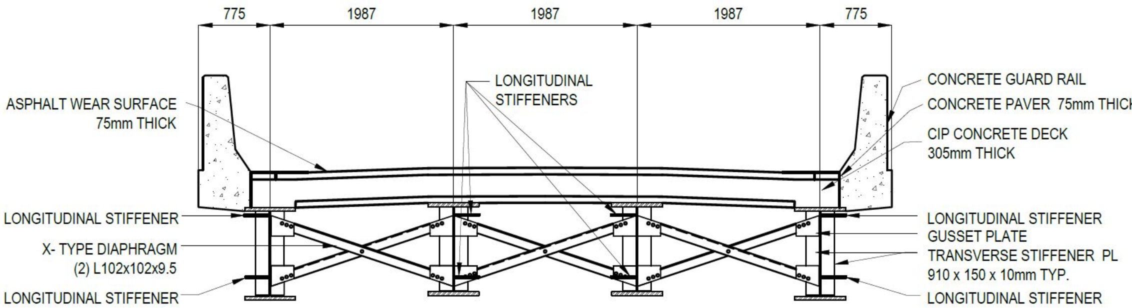

Figure 3-9. Mountbatten Bridge Cross Section (all dimensions mm).

The bridge was in overall excellent condition considering its age and location above a waterway. Minimal rusting and pitting was noted on the main structural members. The galvanized steel decking was in excellent condition as seen in Figure 3-10. Original construction of the bridge used aluminum guardrails, which were replaced with fully integrated concrete guardrails in 2008. The roadway surface was paved asphalt in good condition with no potholes, limited cracking and light wheel track rutting. The main road leading to and from the bridge was hard packed gravel with concrete approach slabs to the bridge. The road was graded between test days to facilitate similar approach conditions for all tests.

Figure 3-10. Underside of the Mountbatten Bridge from East Abutment.

3.4.2.2 Instrumentation

A preliminary analysis was completed using the commercial software CSi Bridge [17] in order to identify the maximum moment locations to be used as the ideal locations for instrumentation. The selected instrumentation locations on each span were representative of where the maximum effects generated by the vehicle loads would be observed. There were three primary locations of instrumentation, installed in the positive moment and the negative moment location of the bridge. Location 1 (L1) was on the West span and at a distance of 13.5 m from the West bearings. Location 2 (L2) was located at a distance *H* (the bridge height of 1.3 m) to the West of the center pier support in the negative moment zone. Location 3 (L3) was on the East span and at a distance of 14.0 m from the East bearings. These areas were instrumented with strain gauges (on girders and steel decking), accelerometers, wired linear position string potentiometers, and a stochastic pattern for Digital Image Correlation (DIC). The main sources of data for this paper are the results obtained from the 10-millimeter 120-ohm strain gauges. Strain gauges were also installed on cross-bracings and stiffeners, and on the steel girders in a 0˚/45˚/90˚ rosette configuration at a distance *H* from the East abutment. Figure 3-11 illustrates the locations of all instrumentation and provides cardinal reference to relate instrumentation locations to the superstructure of the bridge.

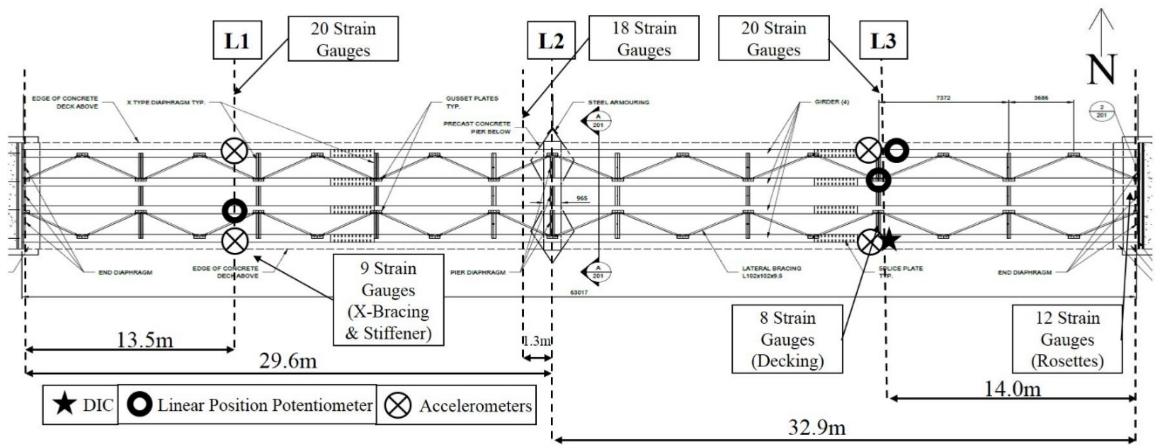

Figure 3-11. Instrumentation Locations.

For the strain gauges installed on the girders in the positive and negative moment locations, the typical strain gauge installation locations are shown in Figure 3-12. Due to site limitations, only one strain gauge was installed on some of the girders in the negative moment location (L2).

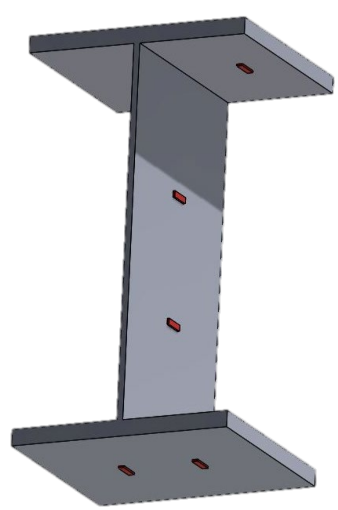

Figure 3-12. Typical Strain Gauge Installation Locations.

The critical data for computing DEF values is a result of the maximum response as observed from the underside of the bottom flange. Therefore, two strain gauges were installed on the bottom flange for redundancy and to increase the data capture. The additional strain gauges on the web and top flange were also installed to guarantee a clear picture of the stress-strain distribution across the depth of the girder, and to maximize obtained data from testing should it be used for future analysis.

All of the instrumentation was connected to three HBM MGCplus data acquisition systems in order to account for the 95 instrumentation channels. The data acquisition systems were controlled by a single computer to synchronize test results into a single file. The data sample rate was 1200 Hertz in order to avoid the risk of any data aliasing during the dynamic tests.

Wired linear position string potentiometers were installed to the underside of the girders. Due to technical difficulties during the testing, several potentiometers malfunctioned, and only three were able to provide displacement data. The potentiometers had their cable half extended, and were then secured to weights that were placed in the water directly below the displacement location. This arrangement provided a measurement range of 150 mm.

Due to the multiple instruments at various locations, instrumentation wire length varied from 4 m to 25 m long. Shielded cable was used in order to assist in minimizing interference from the structure and prevent data loss. In order to verify the quality of the data obtained, the identical setup of data acquisition systems was recreated in a laboratory setting with wire lengths of 1 m and 25 m. Wire length had no significant effect on strain values, where values measured during repeated load tests had an error of less than 0.02%.

3.4.3 Test Program

All test vehicles underwent a very similar test program. All vehicles' static response was obtained along the centerline in both directions, and along the edge of the bridge. It was necessary to capture the static response of vehicles travelling in both directions because, particularly for the wheeled vehicles, the axle positioning affected the bridge response. Each test vehicle crawled (5 km/h) onto the bridge and then stopped at the location that would generate the maximum effects for the instrumentation at L3. The crawling on and off was essential to ensure that the maximum static strain response would be captured even if the position of the stopped vehicle did not exactly capture the maximum static strain [12]. The vehicles did not stop at L1 and L2, and the crawl speed data was used to provide the maximum static response for those locations.

Because smooth surface test results are predominantly used in the development of design code provisions, the smooth surface tests are fundamental in comparing different types of military vehicles in order to provide recommendations for DLA values. All vehicles travelled along the centerline and edge of the bridge from 10 km/h to 50 km/h, at 10 km/h intervals. Sufficient lead distance from the approach slab of the bridge was given to the vehicles so that they would be at the test velocity once they began crossing the bridge. The velocity of vehicle was constant and gear changes were avoided on the bridge. Because similar tests were carried out by both wheeled and tracked vehicles, the results are of key interest for comparing the different dynamic loading effect behaviours between the two vehicle types.

3.4.4 Procedures

Preparation and installation of the instrumentation occurred prior to the planned testing period. All instrumentation was calibrated and verified to be functional. Preliminary data was obtained at the end of the preparation week from various vehicles crossing the bridge and was used to verify the functionality and quality of data obtained from the strain gauges. The bridge behaviour was notably stiffer than expected which is attributed to the added resistance created by the fully composite concrete barriers, which also moved the neutral-axis higher. The higher neutral axis was closer to the top flange of the girder, which generated minimal response in the data at those gauges. An image of the typical strain distribution with girder height is shown in Figure 3-13, which illustrates the distinctly linear distribution with depth of strain results, and contributes to the confidence in the data obtained from the strain gauges. Prior to each vehicle conducting a test, all instrumentation was zeroed and a new data capture sequence commenced in order to eliminate any thermal effects on the strain measurements throughout the testing program.

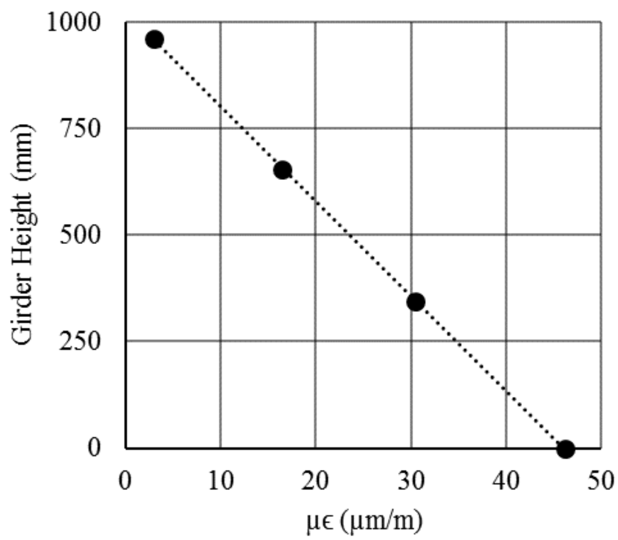

Figure 3-13. Typical Micro-Strain (με) Distribution with Girder Height when Subjected to a Vehicle Load.

The static response of all vehicles at all locations were critical tests and the results of these tests were incorporated into the computation of all DEF values. The static response for all bottom flange strain gauges, gathered from each girder at L1, L2, and L3, for all vehicles travelling in both directions, were graphed in order to review the validity of the maximum response observed. The maximum observed static response at L3 was affected by the braking and acceleration of the test vehicles, demonstrated in Figure 3-14; therefore, it was necessary to appropriately account for these effects in the analysis of the static response. The effects of axle positioning were also reviewed, as particularly for the HLVW, the stationary position did not generate the maximum observed static response. The maximum observed static response at L1 and L2 were from the vehicle crawling across those locations.

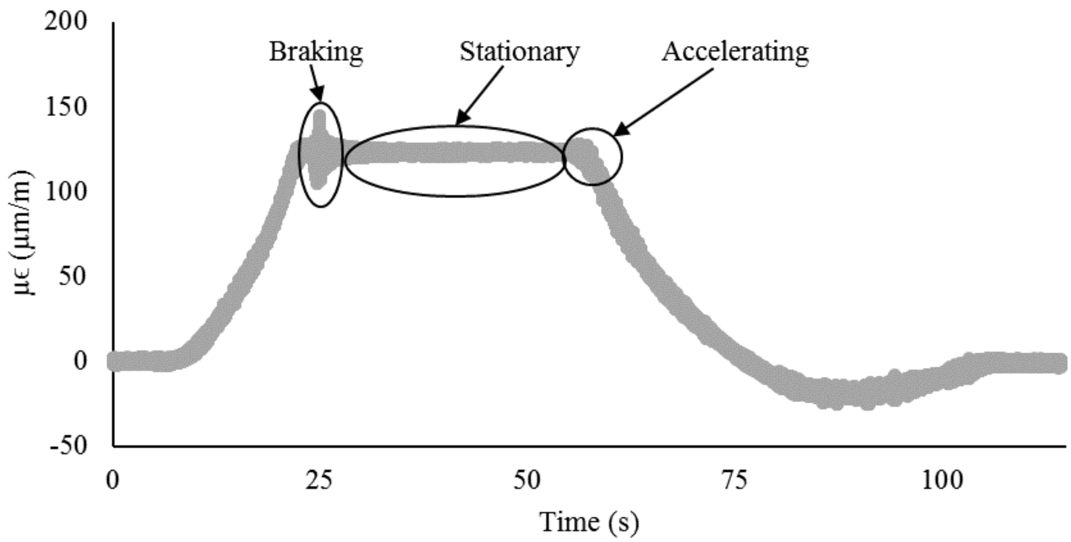

Figure 3-14. Static Response for the Leopard 2 at L3.

The results from all tests were compiled for review and analysis. Once the maximum static responses were validated, they were compared to the maximum dynamic responses for each test, at each location, and for each vehicle using Equation 3-2. As it is appropriate to limit calculated DEF values and in order to assist with mitigating the effects of variations in vehicle positioning, the maximum observed dynamic response was compared to the maximum observed static response resulting in a single DEF value per test for each location.

DEF values from L1 and L3 were obtained from eight strain gauges each, and from L2 the results are from five strain gauges. Each vehicle completed ten tests on a smooth surface; therefore, when DLA values were calculated per vehicle with Equation 3-5, they were a result of 30 DEF values. When grouped into tracked and wheeled military vehicle categories, DLA values were a result of respectively 30 and 90 DEF values.

3.5 Results

Each vehicle demonstrated different behaviour for each smooth surface test. Graphical representations of the calculated DEF values for all vehicles for the smooth surface tests are shown in Figure 3-15 to Figure 3-18. The Leopard 2's DEF values were consistently similar, resulting in a tighter spread of DEF values. All wheeled vehicles' DEF values were individually variable with inconsistent DEF values. While having data from multiple wheeled military vehicles is extremely beneficial, none of the wheeled vehicles were the same mass as the Leopard 2. The HLVW was the closest in mass, at 83% of the Leopard 2, with its mass distributed over five axles. Heavier vehicles tend to report lower experimental dynamic effects [14], which is important to note when comparing the results between vehicles. However, the notably different behaviour cannot be attributed solely to differences in vehicle mass, and is mostly due to the distinct difference in the dynamic behaviour between tracked and wheeled vehicles.

3.5.1 Leopard 2

The Leopard 2 test results were the most consistent among all vehicles used. A summary of the DEF values calculated from each location and for each test completed is illustrated in Figure 3-15. When examining each test individually, the Leopard 2 produced similar results at all locations. This results in a low standard deviation between for the Leopard 2 DEF results, of 0.04. The result of the low standard deviation carries through to generate low DLA values. The maximum observed DEF value of 1.14 was from the negative moment location, L2, when the Leopard 2 was travelling along the edge of the bridge at 50 km/h. On-site observations from testing do correlate to the Leopard 2 test results – the Leopard 2 was consistently smooth and steady across the bridge, with minimal vibrations/oscillations of the bridge deck felt by observers on the bridge. The vibrations appeared to coincide with the linked track of the Leopard 2 making contact with the bridge.

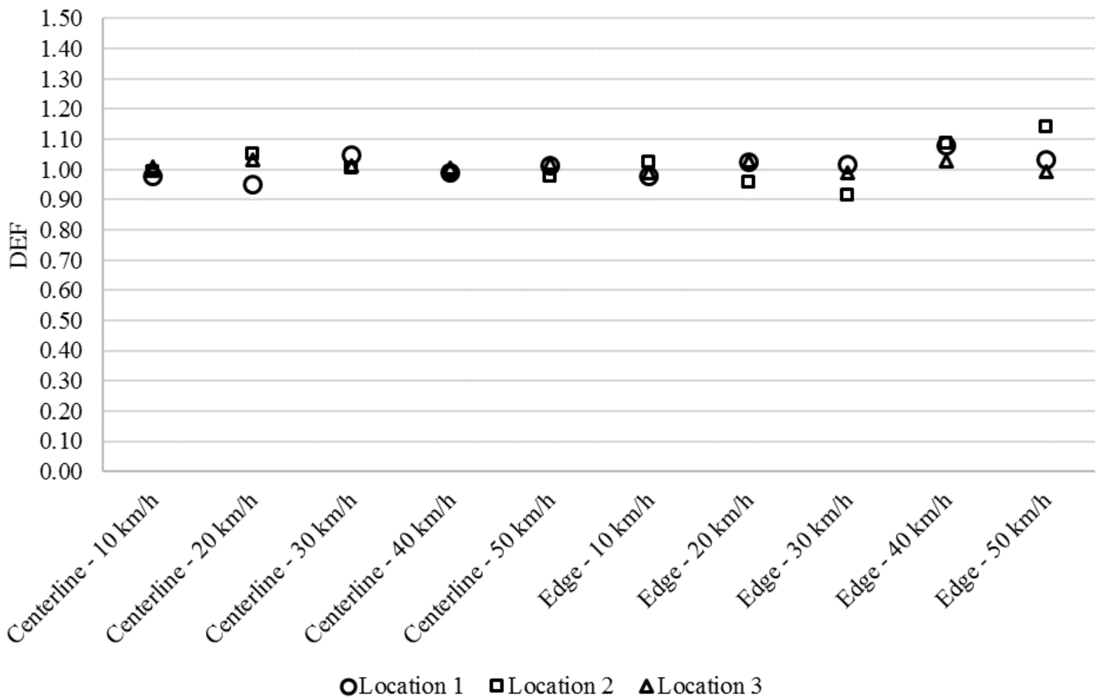

Figure 3-15. Leopard 2 Smooth Surface DEF Values.

3.5.2 HLVW

The HLVW results had some tests with a tighter spread in values, yet the general trend demonstrated significant variability in the data. The HLVW produced the lowest standard deviation amongst the wheeled vehicles of 0.07. The maximum observed DEF value of 1.19 was from the negative moment location, L2, when the HLVW was travelling along the edge of the bridge at 10 km/h. The HLVW was the only vehicle with a trailer, which could have contributed to increased dynamic effects created by the tractor-trailer interaction; however, this effect was minimized with the HLVW being at a steady and constant speed as it travelled across the bridge. Interestingly, there was a tight spread in DEF values at 30 km/h both along the centerline and along the edge of the bridge.

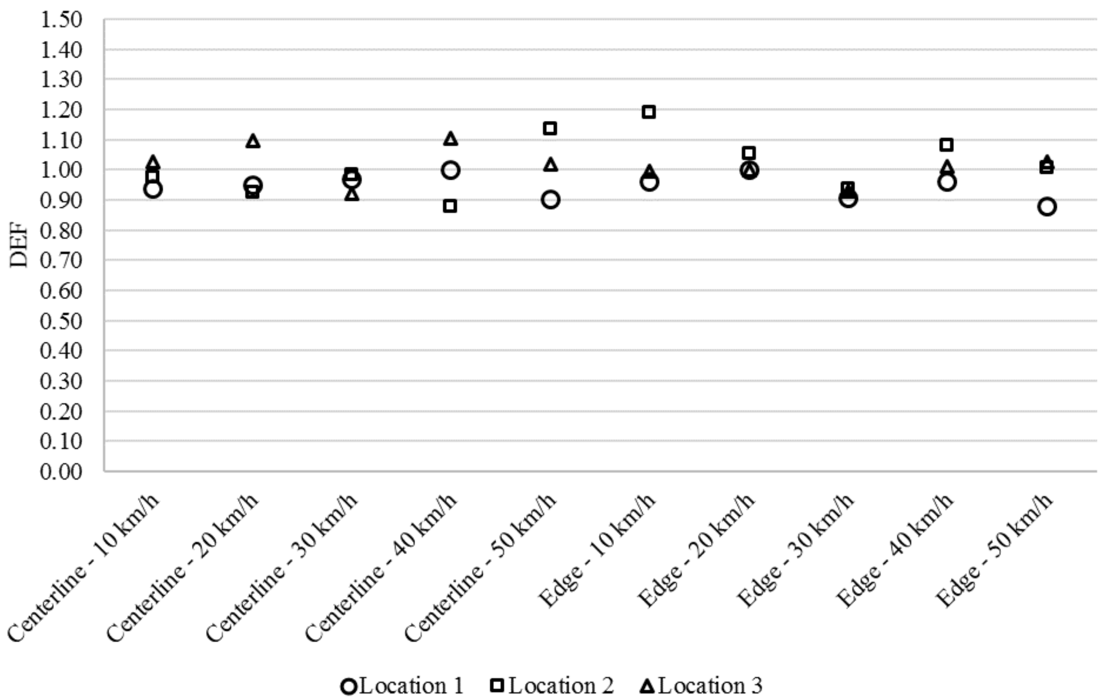

Figure 3-16. HLVW Smooth Surface DEF Values.

3.5.3 E-LAV

The E-LAV results are the most variable amongst all tested vehicles, indicated by its standard deviation of 0.10. The E-LAV also demonstrated the maximum observed DEF value of 1.19 twice, once while it travelled at 50 km/h along the centerline of the bridge, and again at 20 km/h along the edge of the bridge. The E-LAV has four axles contained within the shortest wheelbase of all tested vehicles, which could be a contributing factor in the variability of results. The E-LAV appears to have a consistent spread in its DEF values throughout most tests. L3 is consistently near the top boundary of results for all of the centerline tests, but then trends near the bottom of calculated DEF values for the edge tests.

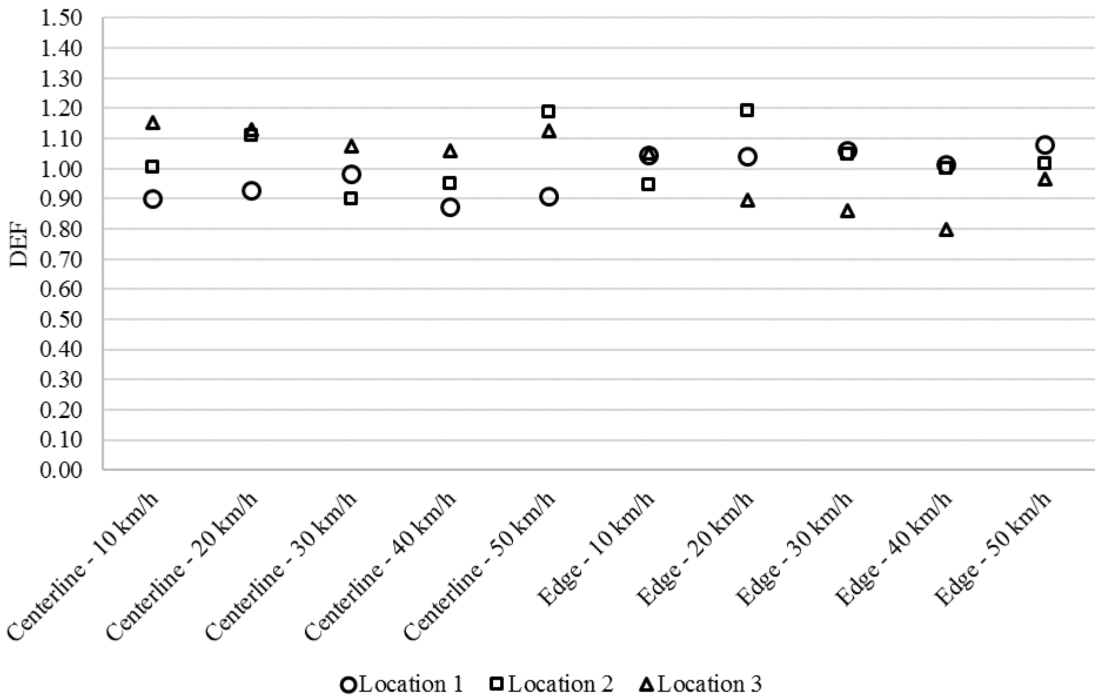

Figure 3-17. E-LAV Smooth Surface DEF Values.

3.5.4 Cougar

The Cougar demonstrated relatively average results amongst the wheeled vehicles with a mean DEF of 0.99 and a standard deviation of 0.08. The Cougar had the least number of axles transferring its load, and it was the lightest vehicle tested. Light vehicles are known to produce higher dynamic amplifications [14], but this was not the case for the Cougar. There was a similar tight spread in DEF values at 50 km/h (fastest test speed) both along the centerline and along the edge of the bridge for the Cougar.

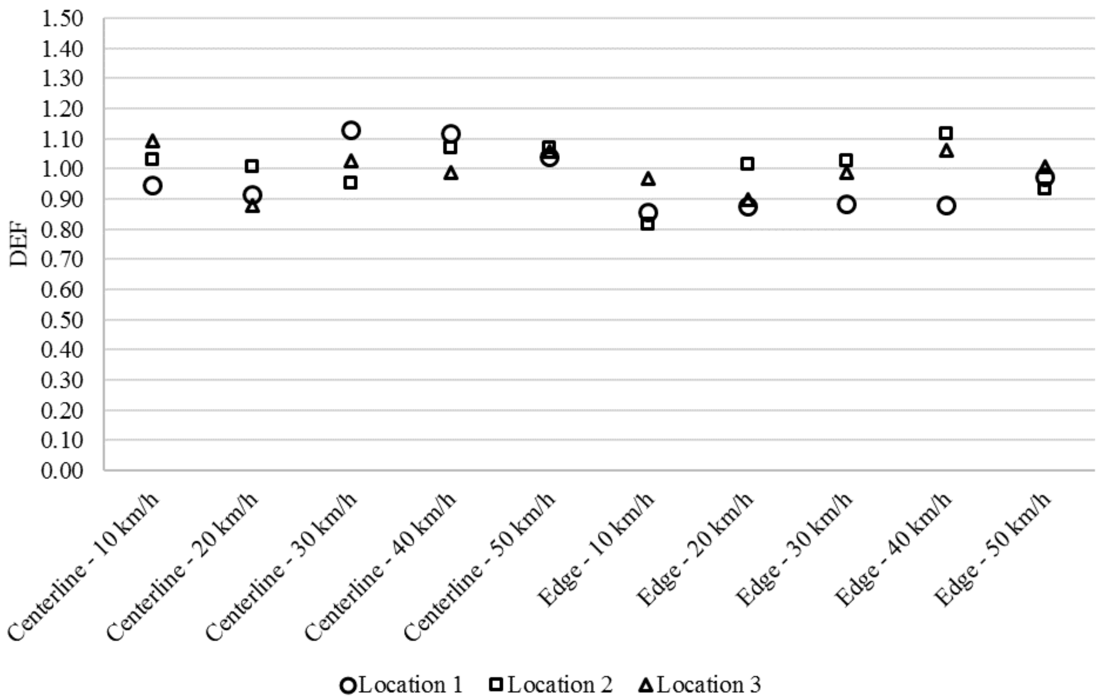

Figure 3-18. Cougar Smooth Surface DEF Values.

3.5.5 Mean DEF Values, Standard Deviation, and DLA Calculation

The mean DEF values and their respective standard deviations (σ) for all vehicles are presented in Table 3-3. The mean DEF values of the wheeled vehicles are all equal to or lower than that of the Leopard 2, yet they all produced a greater standard deviation. The stable and consistent Leopard 2 DEF values, seen in Figure 3-15, are crucial to consider, because that behaviour generates a lower coefficient of variation, which in turn leads to lower calculated DLA values.

Table 3-3. Smooth Surface Mean DEF and Standard Deviation Values.

Vehicle	Mean DEF	DEF σ	Data Points
Leopard 2	1.01	0.04	30
HLVW	0.99	0.07	30
E-LAV	1.01	0.10	30
Cougar	0.99	0.08	30
All Wheeled Vehicles	1.00	0.09	90

As mentioned in Section 3.2.2, the *CSA S6-14 CHBDC* statistical parameters for DLA can be used as a method to assess the validity of test results. To examine how test results perform within two standard deviations, the following expression can be used:

$$[1 + (DLA)(\delta)] \pm [2(DLA)(\delta)(v)] \qquad 3\text{-}6$$

Equation 3-6 produces an upper bound, within two standard deviations, for the *CSA S6-14 CHBDC* mean load method experimental values of 1.39. All of the observed DEF values were below the upper bound of two standard deviations. The maximum observed DEF value of 1.19 occurred three times, when the HLVW travelled at 10 km/h along the edge of the bridge and when the E-LAV travelled at 50 km/h along the centerline of the bridge, and again when the E-LAV travelled at 20 km/h along the edge of the bridge. These results add to the validity of the test results, and the relatively low DEF values support the concept of a reduced DLA value used for military traffic in North American applications of the MLC System.

Multiple combinations of reliability indexes and associated live-load factors were used to calculate DLA values, using Equation 3-5, and the results are shown in Table 3-4. Reliability index and live-load factor provisions were used that include recommendations from Billing (1984), STANAG 2021, *CSA S6-14 CHBDC*, and specific to military traffic categories from MacDonald (2014).

Table 3-4. Smooth Surface DLA Results.

β & α_L Source	β	α_L	DLA				
			Leopard 2	*HLVW*	*E-LAV*	*Cougar*	*All Wheeled*
Billing (1984)	3.50	1.40	0.07	0.10	0.15	0.11	0.12
STANAG 2021	3.30	1.35	0.07	0.10	0.14	0.11	0.12
CHBDC Section 3.2 – Normal Traffic	3.75	1.70	0.06	0.09	0.13	0.10	0.11
CHBDC Table 14.8 – Live Load Factors, Normal Traffic, All Types of Analysis	2.50	1.35	0.05	0.07	0.11	0.08	0.09
	2.75	1.42	0.06	0.08	0.12	0.08	0.09
	3.00	1.49	0.06	0.08	0.12	0.09	0.10
	3.25	1.56	0.06	0.08	0.12	0.09	0.10
	3.50	1.63	0.06	0.09	0.13	0.09	0.10
	3.75	1.70	0.06	0.09	0.13	0.10	0.11
	4.00	1.77	0.06	0.09	0.13	0.10	0.11
CHBDC Table 14.10 – Live Load Factors, Permit PA Traffic – Simplified Analysis – Short Spans	2.50	1.48	0.05	0.07	0.10	0.07	0.08
	2.75	1.55	0.05	0.07	0.11	0.08	0.09
	3.00	1.62	0.05	0.07	0.11	0.08	0.09
	3.25	1.70	0.05	0.08	0.11	0.08	0.09
	3.50	1.78	0.05	0.08	0.12	0.09	0.10
	3.75	1.87	0.06	0.08	0.12	0.09	0.10
	4.00	1.96	0.06	0.08	0.12	0.09	0.10
CHBDC Table 14.10 – Live Load Factors, Permit PA Traffic – Simplified Analysis – Other Spans	2.50	1.28	0.06	0.08	0.12	0.08	0.09
	2.75	1.34	0.06	0.08	0.12	0.09	0.10
	3.00	1.40	0.06	0.08	0.13	0.09	0.10
	3.25	1.47	0.06	0.09	0.13	0.10	0.11
	3.50	1.53	0.06	0.09	0.14	0.10	0.11
	3.75	1.60	0.06	0.09	0.14	0.10	0.11
	4.00	1.67	0.07	0.10	0.14	0.11	0.12
MacDonald (2014)	3.75	1.77		0.08			
	3.75	1.48			0.15	0.11	
	3.75	1.33	0.08				

The calculated Leopard 2 DLA values are consistently 56% to 61% of the DLA values for all wheeled vehicles. As such, it is suggested as appropriate that, irrespective of which code or provisions are used in bridge design or assessment, the DLA used for tracked vehicles should be taken as two-thirds that of wheeled vehicles.

3.6 Code Comparison

Dating back to the late 1920s, a simple expression was used to account for the complex interaction between bridges and their vehicular load and was calculated with the following equation [4]:

$$I = 15/(L + 38) \tag{3-7}$$

where I refers to the impact fraction (not to exceed 0.30), and L the span length in meters.

The current version of the *CHBDC*, *S6-14*, provides more guidance for the application of a DLA depending on the number of axles, and a compressed summary is shown in Table 3-5.

Table 3-5. CSA S6-14 DLA Provisions [6].

Condition	DLA
Deck joints	0.50
Where only one axle of the design truck is used	0.40
Where any two axles of the design truck are used	0.30
Where three axles or more the design truck are used	0.25

Note: all values are multiplied by 0.70 for wood components

A previous version of the *Ontario Highway Bridge Design Code* (*OHBDC 1983*) used the first flexural frequency of a bridge in order to establish the relationship of which DLA value to use, shown in Figure 3-19 [4].

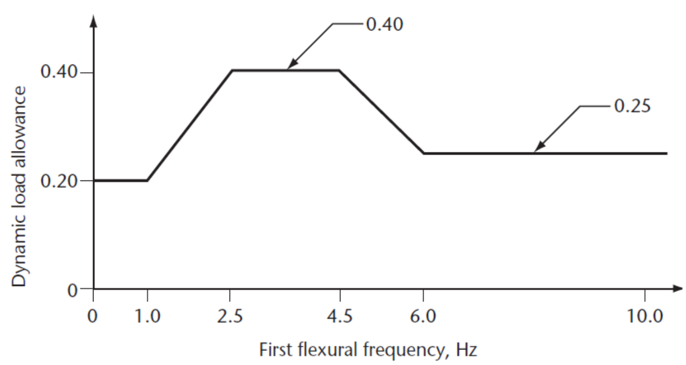

Figure 3-19. DLA/Frequency Relationship [4].

All countries implement a form of a DLA; however, the parameters controlling implementation tend to vary between span length, fundamental frequency, bridge type, or bridge component. Deng et al. (2015) provided a comprehensive overview that reviews the various provisions of several design code values for DLA from the United States of America (USA), China, New Zealand, European Union (EU), Britain, and Japan. Each country varies their calculation of a DLA, and application of a DLA value has generally been simplified despite dynamic effects being the result of many factors [11]. Equation 3-7, from the 1920s, is the most prevalently used calculation for DLA; it is a function of bridge span length and there are similar implementations in

1992 AASHTO, 1989 Chinese design code, and 2013 New Zealand design code for spans greater than 12 m. The lack of consistency between national design codes for wheeled vehicles indicate that the dynamic effects generated by vehicles are still not fully understood nor in consensus between nations.

Parameter values corresponding to the Mountbatten Bridge, as shown in Table 3-6, were used for all calculations:

Table 3-6. DLA Parameter Values.

Parameter	Value
Span Length (m)	32.9
Fundamental Frequency (Hz)	2.5
Bridge Type	Steel
Number of Axles	Varies
Number of Traffic Lanes	Single and Two
Traffic Loads	Normal
Bridge Component	All Other (not Deck Joints)

Recommended DLA values for respective design provisions and the results are shown in Table 3-7. Where a design code uses a DLF or DAF term, Equation 3-3 was applied in order to compare the results as DLA values.

Table 3-7. Various Design Code DLA Values.

Origin	Equation	DLA Value
Canadian 2014 S6-14 CHBDC Commentary [4]	$I = 15/(L + 38)$	0.21
Canadian 2014 S6-14 CHBDC [6]	Prescribed in code	0.25
Canadian 1983 OHBDC [4]	See Figure 3-19	0.40
US AASHTO 1992, 1996, and 2002 Standard Specification for Highway Bridges [18] [19] [20]	$IM = \frac{15.24}{L + 38.10} \leq 0.3$	0.21
US AASHTO 1998 and 2012 LRFD bridge Design Specifications [21] [22]	Prescribed in code	0.33
Chinese 1989 General Code for Design of Highway Bridges and Culverts [23]	$IM = \frac{15}{37.5 + L}$	0.21
Chinese 2004 General Code for Design of Highway Bridges and Culverts [24]	$IM = 0.1767lnf - 0.0157$ $when\ 1.5\ Hz \leq f \leq 14\ Hz$	0.15
New Zealand 2013 Bridge Manual [25]	$DLF = 1 + \frac{15}{L + 38}\ when\ L > 12\ m$	0.21
EU 2003 Eurocode 1: Actions on Structures – Part 2: Traffic Loads on Bridges [26]		
- *Single Lane*	$DAF =\ 1.4L\ when \geq 15\ m$	0.40
- *Two lane*	$DAF = 1.3 - \frac{0.4}{100}L\ when\ L \leq 50\ m$	0.17
British 2006 BS 5400-2, Steel, Concrete and Composite Bridges. Part 2: Specification for Loads [27]	Prescribed in code	0.25
Japanese 1996 Specifications for Highway Bridges. Part 1: Common Specifications [28]	$IM =\ 20/(50\ +\ L)$	0.24
Canadian B-GL-361-014FP-001 MANUAL FOR MILITARY NONSTANDARD FIXED BRIDGES [3]	Prescribed in document	0.15
US FM 3-43.343 Military Nonstandard Fixed Bridging [8]	Prescribed in document	0.15

The greatest prescribed DLA values of 0.40 calculated were from the *1983 OHBDC* as a result of the bridge frequency, and from the EU 2003 Eurocode for single lane traffic as a result of the bridge span length. The maximum calculated DLA value was 0.12 for All Wheeled Vehicles, which falls within all of the examined design code provisions. While some design code provisions do provide what appear to be very conservative DLA values, they are from previous editions that have since been replaced, or were generated from a non-calculated value prescribed as a function of bridge span length. Keeping in mind that these DLA values are specific to wheeled traffic, it may be appropriate to reduce the various DLA values by one-third for tracked vehicle analysis in accordance with the conclusions presented in Section 3.5. All nations would benefit from a reduced DLA by one-third for tracked vehicles, effectively increasing their existing bridge capacity by 5% to 13% for tracked vehicles.

3.7 Conclusions

The experimental program discussed in this paper was carried out in order to examine the difference in dynamic behaviour between wheeled and tracked military vehicles, to examine the difference in calculated DLA values for different classes of military traffic, and to examine how experimental DLA results compare against various nations' design codes. The results from this experimentation should be of interest to all military and defence team engineers because the demonstrated difference in dynamic behaviour between tracked and wheeled vehicles could represent considerable increases in existing bridge infrastructure load capacity ratings. A summary of the key results and observations are presented below:

1. Tracked and wheeled vehicles exhibit distinctly different dynamic behaviour.

2. Out of the wheeled vehicles testing program, the E-LAV test data consistently produced higher DLA values due to its variable dynamic behaviour.

3. The STANAG 2021 recommended reliability index and live-load factor values result in a calculated DLA for wheeled vehicles of 0.12. This value is less than the current recommended DLA of 0.15 for military traffic. Therefore, these results are supportive of the reduced DLA value, of 0.15, for military traffic.

4. Observed and calculated DLA values for the Leopard 2 and all wheeled vehicles as a result of experimentation fell within all examined design code DLA provisions.

5. The Leopard 2 DLA values are consistently less than two-thirds of the DLA values for all wheeled vehicles.

6. It may be appropriate to reduce design code provisions of DLA values used for wheeled traffic by one-third for tracked vehicles, which would represent a significant strategic increase in the mobility of tracked vehicles.

7. Application of a reduced DLA for tracked vehicles could result in an increase to existing bridge capacity of 5% to 13% for tracked vehicles.

8. More research between the dynamic effects of tracked versus wheeled vehicles should be carried out for various other tracked and wheeled vehicles and on different bridge types, in order to further support the recommendation of a reduced DLA for tracked vehicles.

3.8 Acknowledgements

This research was funded by the Department of National Defence and would not have been possible without the support and cooperation of several Canadian Armed Forces directorates and units, specifically 2 Combat Engineer Regiment, Director Combat Support Equipment Management, and Director Armament Sustainment Program Management.

3.9 References

[1] *NATO Standardization Agreement 2021 - Military Load Classification of Bridges, Ferries, Rafts and Vehicles (Edition 8),* NATO, 2015.

[2] A. MacDonald, "Applying Probabilistic Methods to the NATO Military Load Classification System for Bridges," Western University Master's Thesis, London, 2014.

[3] DND/MDN Canada, *B-GL-361-014FP-001 MANUAL FOR MILITARY NONSTANDARD FIXED BRIDGES (DRAFT),* Ottawa, 2008.

[4] CSA Group, *Commentary on CSA S6-14, Canadian Highway Bridge Design Code,* Mississauga: CSA Group, 2014.

[5] R. Lenner, M. Keuser and M. Sykora, "Assessment of existing reinforced concrete bridges exposed to military loads," in *Novák and Vořechovský: Proceedings of the 11th International Probabilistic Workshop*, Brno, 2013.

[6] CSA Group, *S6-14, Canadian Highway Bridge Design Code,* Mississauga, Ontario: CSA Group, 2014.

[7] R. Lenner, "Safety Concept and Partial Factors for Military Assessment of Existing Concrete Bridges," Universität Der Bundeswehr München, Munich, 2014.

[8] Department of the Army, *FM 3-43.343 Military Nonstandard Fixed Bridging,* Washington, 2002.

[9] A.J. MacDonald, R.G. Wight and F.M Bartlett, "Probabilistic Gross Vehicle Weights and Associated Axle Loads for Military Vehicles in Bridge Evaluation and Code Calibration," *Advances in Military Technology,* vol. 12, no. 1, pp. 129-145, 2017.

[10] A.J. MacDonald, R.G. Wight and F.M Bartlett, "Acceptable Risk in Military Bridge Evaluation," *Advances in Military Technology,* vol. 11, no. 2, pp. 197-209, 2016.

[11] L. Deng, Y. Yu, Q. Zou and C.S. Cai, "State-of-the-Art Review of Dynamic Impact Factors of Highway Bridges," *Journal of Bridge Engineering,* vol. 20, no. 5, pp. 04014080-1 to 04014080-14, 2015.

[12] P. Paultre, J. Proulx and M. Talbot, "Dynamic Testing Procedures for Highway Bridges Using Traffic Loads," *Journal of Structural Engineering,* vol. 121, no. 2, pp. 362-376, 1995.

[13] B. Bakht and S.G. Pinjarkar, *Dynamic Testing of Highway Bridges - A Review,* Washington: Transportation Research Record 1223, Transportation Research Board, 1989, pp. 93-100.

[14] R. Shepherd and R.J. Aves, "Impact Factors for Simple Concrete Bridges," in *Proc. Inst. Civ. Eng., Vol. 55 Part 2, Research and Theory, Paper 7548*, 1973.

[15] J. Billing, "Dynamic Loading and Testing of Bridges in Ontario," *Canadian Journal of Civil Engineering,* vol. 11, no. 4, pp. 833-843, 1984.

[16] R. Cantieni, "Dynamic Load Tests on Highway Bridges in Switzerland: 60 Years Experience of EMPA," Swiss Federal Laboratories for Materials and Testing Research, Dubendorf, 1983.

[17] Computers and Structures, Inc., *CSi Bridge,* Walnut Creek, CA: v20.1.0, 2018.

[18] AASHTO, "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials, Washington, 1992.

[19] AASHTO, "Standard Specifications For Highway Bridges," American Association of State Highway and Transportation Officials, Washington, 1996.

[20] AASHTO, "Standard Specifications for Highway Bridges," American Association of State Highway and Transportation Officials, Washington, 2002.

[21] AASHTO, "LRFD Bridge Design Specifications," American Association of State Highway and Transportation Officials, Washington, 1998.

[22] AASHTO, "LRFD Bridge Design Specifications," American Association of State Highway and Transportation Officials, Washington, 2012.

[23] Ministry of Transport of the People’s Republic of China (MTPRC), "General Code for Design of Highway Bridges and Culverts," Beijing, 1989.

[24] Ministry of Transport of the People’s Republic of China (MTPRC), "General Code for Design of Highway Bridges and Culverts," Beijing, 2004.

[25] New Zealand Transport Agency (NZTA), "Bridge Manual," Wellington, 2013.

[26] European Committee for Standardization (CEN), "Eurocode 1: Actions on Structures—Part 2: Traffic Loads on Bridges," Brussels, 2003.

[27] British Standards Institution (BSI), "Steel, Concrete and Composite Bridges. Part 2: Specification for Loads.," London, 2006.

[28] Japan Road Association (JRA), "Specifications for Highway Bridges. Part 1: Common Specifications," Tokyo, 1996.

4. MANUSCRIPT #2: "DYNAMIC LOADING EFFECTS OF WHEELED AND TRACKED MILITARY VEHICLES ON A BRIDGE FROM SURFACE IRREGULARITIES AND BRAKING"

4.1 Abstract

The difference in dynamic loading effects between wheeled and tracked military vehicles was examined through field-testing of a bridge that included vehicles passing over obstacles, and conducting braking. There is a dearth of guidance in current bridge design or assessment codes to differentiate between the dynamic loading effects of wheeled and tracked vehicles, not only on smooth bridge decks, but also on uneven surfaces. The North Atlantic Treaty Organization (NATO) utilizes a Military Load Classification (MLC) System that compares the load effects of vehicles to the capacity of bridges in order to determine the feasibility of crossing. The MLC System uses the same dynamic loading effects values for all vehicle types, which may be limiting the mobility of tracked vehicles in military situations.

Obstacles and debris on bridge deck surfaces are likely to be encountered during military conflict or post-disaster situations. These surface irregularities are seldom accounted for in bridge design or assessment codes. A comprehensive test program was carried out for Canada's main battle tank, the Leopard 2, in order to examine the dynamic loading increase when it travelled over obstacles, and a marked increase in the tank's dynamic loading effects was noted. A similar yet reduced test program was carried out using wheeled vehicles, and those results demonstrated considerable increase in dynamic loading effects when compared to the Leopard 2. The research indicates a significant increase in dynamic effects can occur when a vehicle travels over an obstacle. In some instances, the dynamic loading effects generated by wheeled vehicles were approximately five times that of the Leopard 2. All vehicles also carried out a series of tests in which they decelerated from various speeds to a full stop as rapidly as possible. Design codes generally apply a braking force separately from the DLA, and longitudinally at or near the bridge deck. However, this research indicates an increase in dynamic effects when braking occurs, particularly over short distances.

4.2 Introduction

4.2.1 Military Load Classification System

Military commanders must complete the critical task of route identification and selection when planning missions and operations. During route selection, a process determining which bridges can support the required military vehicle traffic is required. It is essential that the loading effects and bridge capacities are understood and quantified, as armed forces will undoubtedly be required to cross civilian bridges in the conduct of exercises and operations. The North Atlantic Treaty Organization (NATO) utilizes the Military Load Classification (MLC) System outlined in NATO Standardization Agreement (STANAG) 2021 Edition-8 to calculate the MLC of bridges and vehicles. The aim of STANAG 2021 is to provide NATO forces with a standard method for computing the MLC representing the load capacities of all bridges, military ferries and rafts, and the loading effects of military vehicles [1].

MLC values for vehicles are determined by calculating the governing shear and moment loading effects for a convoy of that vehicle type, on spans ranging from 1 m to 100 m. These results are then compared to theoretical standard MLC vehicle loading effects on the same range of spans. This process produces a correlation plot of vehicle MLC values relative to span for both moment and shear. The maximum MLC value determined by either moment or shear at any value of span governs the MLC value for that vehicle. The benefit of analyzing vehicles through this method, vice using its number of axles and Gross Vehicle Weight (GVW), is that it permits each vehicle to be rated on the nominal maximum loading effects that it generates on a simple span [2].

On operations, existing bridges should always be used first, with the goal of protecting the bridges from overloading and possible damage or failure [3].To assign an MLC capacity rating to a bridge, it is necessary to determine the allowable shear and moment capacity of that structure. Once the allowable shear and moment values are determined, the moment and shear capacity for the span is expressed in terms of a factored standard MLC vehicle. Because the load effects of wheeled and tracked military vehicles are unique and different from normal civilian traffic [2], it is appropriate to account for this in determining MLC capacities of bridges by applying suitable factors for military traffic.

Because of the nature of military operations, the level of acceptable risk may be much higher than would be acceptable in non-military peacetime situations. The MLC system described in STANAG 2021 establishes three crossing conditions of Normal, Caution, and Risk Crossing related to the urgency of crossing and the level of acceptable risk. Normal Crossing of a bridge permits unrestricted use of civilian bridges and normal accepted restrictions for any military bridges. Tactical or emergency situations may arise where higher MLC vehicles are required to cross certain bridges. With more controlled crossing criteria and/or the use of less stringent safety criteria, a Caution or Risk Crossing may be followed. A Caution Crossing involves vehicles driving along the centerline of the bridge, guided, at speeds not to exceed 5 km/h, and braking, accelerating, or changing gears is prohibited. The same safety factors will apply as a Normal Crossing, except no Dynamic Load Allowance (DLA) will be used in the capacity calculations. The vehicle crossing constraints remain the same for Risk Crossing, with all safety factors being revised. The increased probability of bridge failure in a Risk Crossing is accepted, along with the understanding that structural stresses may be near the yield limits, but should not exceed the ultimate limit [1]. The construct of this research project represents situations that could be considered as a Caution or Risk Crossings.

4.2.2 Load Factors

To safely assess a bridge's capacity, factors are applied to decrease the resistance of the bridge and increase the effects of the loads. The live-load capacity of a bridge can be estimated by determining the resistance of the structure and then subtracting the effects of the dead load. These adjustments account for a variety of issues including the variability of material properties, quality of construction, and the actual effects of traffic, and are typically determined using reliability-based methods at the ultimate limit state. The live-load factor specifically accounts for unknown overloads during a bridge's lifetime and uncertainties in the load analysis [4]. Although STANAG 2021 is established to provide NATO forces with a standard method for determining MLC values, no specific method is imposed and each country is permitted to use their own procedures and safety factors [1] [5]. Therefore, STANAG 2021 has moved towards the development of a safety concept while allowing individual countries to follow their civilian codes while establishing national standards [1]. Canadian and United States military doctrine suggest a DLA of 0.15 for MLC application [3] [6], and a DLA of 0.25 would be used in *CSA S6-14 CHBDC* application for bridges of moderate span.

4.2.3 Dynamic Load Allowance

A dynamic factor is generally applied to most bridge components to account for the phenomenon that occurs as vehicles cross bridges and generate a dynamic interaction. *CSA S6-14 CHBDC* defines DLA as "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 [7]." Similar terms are used in different nations' literature and design codes including: Dynamic Impact Factor (IM) [8], Dynamic Amplification Factor (DAF) [8] [9], or Dynamic Load Factor (DLF) [8].

The term DLA is used in Canadian design and assessment code implementation. However, when reporting the results of experimentation as a result of practical field testing, Dynamic Effects Factor (DEF) is used. Dynamic Amplification (DA) can be also used to represent the results from experimental testing [9]. Equation 4-1 is used to calculate DEF values and shows the relationship with the DA term.

$$DEF = \frac{R_{dyn}}{R_{sta}} = (1 + DA) \qquad 4\text{-}1$$

where R_{dyn} is the maximum dynamic response of the bridge, R_{sta} is the maximum static response of the bridge, and DA is the abbreviation for Dynamic Amplification.

Several factors can effect the dynamic interaction between bridges and the crossing vehicle loads including the vehicle type, the vehicle weight, the vehicle position with respect to a reference point, the surface condition, and multilane loading [10]. Lighter vehicles tend to demonstrate a higher DEF in testing [11] [12], and that trend should be similar when testing with surface irregularities; however, there is no indication as to how vehicle mass will effect the dynamic effects from braking. The quality of the riding surface of both the bridge wear surface and the bridge approach can have significant influence on the dynamic magnification load effects on the bridge [10]. The condition of the approach has been shown to cause significant dynamic effects as an approach in poor condition can create large initial oscillations that may exceed design code values for DLA [8]. Every bridge and vehicle presents unique parameters, which makes practical field-testing the ideal solution for understanding the dynamic effects generated by vehicles travelling over bridges.

Bridge surfaces are assumed to be smooth in design and with proper maintenance they should remain relatively smooth. Testing for dynamic effects generated by the vehicle bridge interaction is commonly focussed on bridges with "as-is" conditions which should be relatively smooth. However, it is possible for the accumulation of snow/ice or an object falling off a vehicle to create an unplanned irregularity in the riding surface [10]. Yet in a military context, situations may arise where vehicles must stop suddenly and/or travel across bridges that are damaged or have obstacles placed across them. Natural disasters may also cause damage to structures or cover bridge roadway surfaces with debris. Understanding the effects generated by these situations are important for everyone, including vehicle operators and engineers, to grasp in order to prevent overloading of bridges. Depending on the situation, it may be appropriate to follow a Caution or Risk Crossing procedure as described in Section 4.2.1 in a military situation.

4.2.4 Dynamic Testing With Surface Irregularities

As detailed in Section 4.2.3, testing for dynamic effects of the vehicle bridge interaction in order to generate code provisions for DLA is generally focussed on a smooth surface or testing bridges with "as-is" conditions. The approach of placing a wooden plank or other obstacle on the travel path to increase the dynamic effects observed is recognized by researchers, but is not used in all dynamic testing. As noted, an object falling off a vehicle or the accumulation of snow/ice can create a sudden irregularity in the riding surface [10]. While during peacetime military exercises, one would not expect there to be deliberate obstacles on the riding surface, such situations may be encountered during military operations. Therefore, the testing of dynamic amplification over obstacles carries merit in testing with military vehicles.

The use of artificial surface irregularities in order to examine the dynamic interaction between bridges and vehicles has been seldom carried out; even with the consensus among researchers that surface irregularities will cause an increase in dynamic effects. Bridge surface conditions are assumed to be as smooth as possible in design, and proper maintenance is carried out to insure the roadway surface does remain as smooth as possible. A study was carried out during 1976 in Britain to examine the dynamic effects generated by a test vehicle travelling over planks, in order to compare it to the maximum permissible surface irregularity height. Data from testing over obstacles of 20 mm, 30 mm, and 40 mm was extrapolated and determined that an obstacle of 9 mm would generate a peak DEF of 1.4 [13]. These results assist in demonstrating the significant increase in dynamic effects that can occur from relatively small surface irregularities.

Chapter 14 of *CSA S6-14 CHBDC*, which is focussed on evaluation of structures, is conditioned to include a bridge inspection carried out to the satisfaction of the examiner [7]. The *CSA S6-14 CHBDC Commentary* references the 1991 *Ontario Structure Inspection Manual* for details on bridge inspection practice and the deformation one could expect to find [4]. All indications for inspections and noted deficiencies are from a maintenance perspective. Modification factors are provided for the material resistance of components that have experienced defects and deterioration [7]; however, they do not indicate suggested modifications to DLA as a result of surface irregularities. Perhaps as Canadian design codes underwent significant research into validating DLA provisions in 1980 [12], there is belief that it accurately captures bridge "as-is" conditions. Understanding the effects that surface irregularities could have on the dynamic effects generated by the vehicle-bridge interaction assist in reinforcing the importance of maintaining a smooth bridge surface.

Interestingly, all bridges in Latvia undergo dynamic load tests prior to bridge commissioning that include vehicles travelling over smooth and uneven roadway conditions [14]. The uneven pavement tests are representative of damaged pavement or ice bumps and are created by timber planks approximately 50 mm tall and 100 mm wide placed on the bridge wearing surface. Planks are generally 3.0 m to 3.5 m apart over 2/3 of the span length [14]. These represent significant obstacles considering the height and repeated pattern. A study of results from bridge load tests from 1991 until 2012 notes, as one might expect, scattering in DEF results and that bridge type appears to be more controlling in the dynamic responses observed vice span length as seen in most design code provisions [14]. The use of timber/lumber planks as obstacles in a repeated pattern was chosen as the method for creating artificial surface irregularities in this research project.

Situations may arise in military operations where bridges have been damaged or deliberate obstacles have been place across them. Military route denial with timber obstacles is generally focussed on roadways; yet, their use could easily be modified for use on bridges when the intent is slow or deter movement across a bridge. US Army doctrine provides guidance for the construction of log hurdles (shown in Figure 4-1) to slow tank movement, and indicates that obstacles 254 mm in diameter would not be sufficient to stop a tank [15].

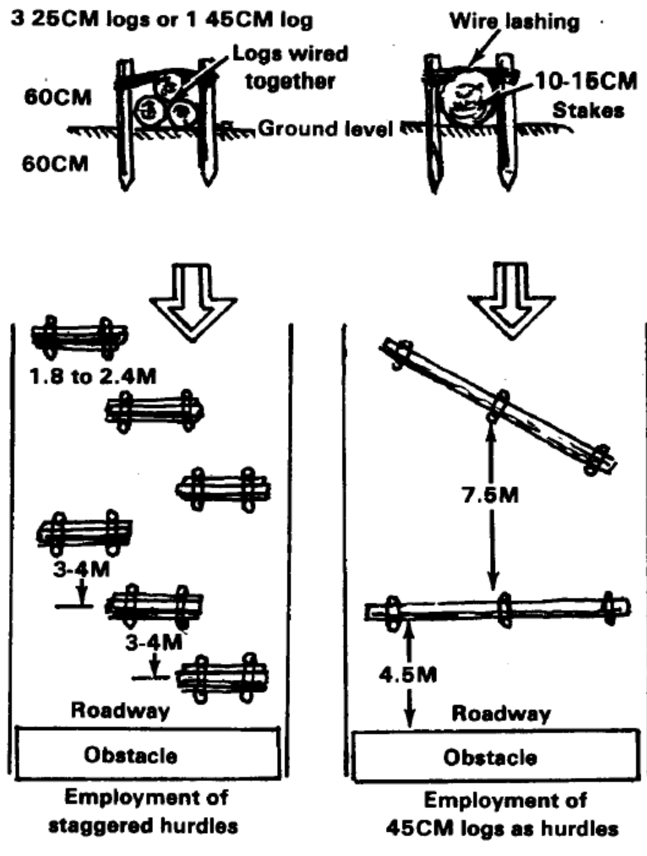

Figure 4-1. Example of Log Hurdles [15].

In the unfortunate event that a natural disaster occurs, it is possible for debris or damage to be present on bridge surfaces. It is necessary to clearly understand the magnitude of dynamic load increase to structures generated by surface irregularities (be they surface roughness, combat damage, deliberate obstacles, or natural disaster debris) to avoid the risk of overloading should vehicles have to traverse any irregularity. This research project included military vehicles conducting exploratory testing into the dynamic effects generated by surface irregularities by using timber planks to generate increased dynamic effects.

4.2.5 Braking Effects

Vehicles will apply a transient longitudinal force to the top of the bridge when braking, which causes a longitudinal motion in the superstructure that is restrained by support reactions [4]. Many factors must be considered in the analysis of braking force including the coefficient of friction between the tires and the wearing surface, and dynamic characteristics of: (1) a bridge's superstructure; (2) the vehicle; and (3) bridge bearing and piers. *CSA S6-14 CHBDC* has simplified the braking effects for ease of design and outlines specifications for how the braking force is to be considered:

> "Braking force shall be considered only at the ultimate limit states.
> Braking force shall be an equivalent static force of 180 kN plus 10% of the uniform distributed load portion of the lane load from one design lane, irrespective of the number of design lanes, but not greater than 700 kN in total.
> The braking force shall be applied at the deck surface" [7].

The simplified application presented in Canadian design code only applies braking as a transient longitudinal force. However, when a vehicle brakes, there is a shift in the load between the axles, changing the vertical forces, and resulting in increased flexural forces. *CSA S6-14 CHBDC* does acknowledge that braking effects will cause dynamic increases, as it is assumed that the sudden application of brakes is unlikely in permit situations that have applied a reduction in DLA [4]; however, the DLA provisions give no indication that they include the dynamic effects as a result of braking. It is further specified that DLA "is not required for centrifugal, braking, collision or pedestrian loads" and that braking is considered a rare event likely to not occur to overloaded vehicles [4].

The National Cooperative Highway Research Program (NCHRP) conducted research into determining the maximum braking force as a fraction of the vehicle weight and determined it to be approximately 25% [4]. The NCHRP value was determined using a stopping distance of 122 m from a speed of 88.5 km/h [4]. Yet research has shown that a shorter braking duration can significantly increase the braking effects [16]. Research into the interaction between vehicle braking and the dynamic increase appears limited. As such, this research project included vehicles coming to a full stop as rapidly as possible to determine the dynamic effects generated from rapid braking, and to examine how they correspond to design code provisions for dynamic allowances.

A similar process that was used to calculate DEF values will be used to calculate the effects from braking. The term Braking Effects Factor (BEF) will be used to denote the strain increase as a result of braking and will be calculated using Equation 4-2:

$$BEF = \frac{R_{brake}}{R_{sta}} \qquad \text{4-2}$$

where R_{brake} is the maximum dynamic response of the bridge as a result of braking and R_{sta} is the maximum static response of the bridge.

4.2.6 Testing Theory and Result Calculations

Both deflection and strain measurements are used to measure the dynamic increase of a structure; therefore, the use of the term static or dynamic response is used in referring to the results obtained. The static and dynamic responses obtained in the conduct of this experiment are a result of strain measurements. In the calculation of both DEF and BEF values, static response results are required. Several methods exist to calculate the static response of a structure. The results presented in this paper were obtained from one instrumented cross-section of the bridge as that is where the surface irregularities and braking occurred. The static response for this experimentation was obtained by each test vehicle crawling onto the bridge at a low speed (less than 5km/h) and then remaining stationary over the instrumented location. The stationary positions may not have produced the maximum static responses due to axle positioning; therefore, the combination of a crawl and stationary test assisted in validating the stationary results. The data from the crawl and stationary tests were graphically examined in order to confirm that the maximum values obtained were not a result of braking or accelerating at the selected stationary position.

Research has shown that the DEF values obtained in testing will be higher at a reference point away from the load than those from a reference point directly below the load [10]. When capturing the static load response of a bridge, the structural members directly under the applied load will carry an increased amount of load than those structural members not directly under the applied load. Yet those structural members located away from the applied load will result in higher DEF values due to their low static response, this effects may be particularly apparent when testing with obstacles and braking. One method for determining the relevance of instrumented locations involves looking at instrumentation contained within its zone of direct influence, which is essentially anything within 45˚ of the test vehicle wheel path [17]. This may be inappropriate for all bridge types due to variations in transverse load distribution.

As previously indicated, vehicle positioning and selected instruments may provide different DEF values at the same cross-section. In order to avoid overestimation of DEF values [9], an option is to divide the largest dynamic response by the largest static response from different instrumented locations at that cross-section. This would result in a single DEF value per test, per instrumented cross-section. Comparing the observed maximum values will also assist in accounting for error in lateral vehicle positioning during the dynamic tests. This method of comparing maximum values from a cross-section was used to calculate the findings for this research project.

Experimental results can undergo a calibration process in order to generate them as design values. This process requires the use of the mean, the coefficient of variation, and chosen provisions for live-load factor and reliability index [10]. The results from Chapter 3 indicate that the DLA applied to tracked vehicles can be conservatively taken as two-thirds of that applied to wheeled vehicles in normal situations. This could be accomplished with the results from this research project; however, the number of tests completed for this portion was minimal which would lead to few results in the determination of mean and coefficient of variability values. Therefore, only the calculated DEF and BEF values were used in comparison and examination of the dynamic loading effects.

4.3 Research Objectives and Scope

This research paper is part of a larger research project that is also looking at developing live-load and DLA values for military vehicles, load distribution of compositely acting bridges, and further analysis on the effects of military traffic. The findings presented in this paper are a result of testing on one bridge with four different military vehicles (one tracked and three wheeled), varying from 26 to 61 tonnes. The objective of this portion of the research is to examine the dynamic effects generated by wheeled and tracked military vehicles braking and passing over surface irregularities on a bridge.

The static response of the bridge was obtained for all vehicles at the locations where they would complete dynamic tests (the centerline for all vehicle and at the edge position of the bridge for the tracked vehicle). Testing followed three categories of: smooth surface passes, passes with obstacles across the bridge deck, and passes incorporating braking. The results from the smooth testing were used to generate DLA values and are reported in Chapter 3. Vehicles conducted tests at varying speeds along the centerline position of the bridge for both braking and over surface irregularities. Obstacle and braking DEF values were calculated from the results of the longer, more critical, span as that is where the obstacles were positioned and the braking occurred.

4.4 Experimental Program

4.4.1 Vehicles

Three different wheeled and one tracked military vehicles were used for the experimental program. The tracked vehicle used is shown in Figure 4-2, and is Canada's main battle tank (MBT), the Leopard 2. The three wheeled vehicles used in testing included: a Heavy Logistics Vehicle Wheeled (HLVW) with a low-bed shown in Figure 4-2, an Engineer Light Armoured Vehicle (E-LAV) shown in Figure 4-4, and an Expedient Route Opening Capability (EROC) vehicle know as the Cougar shown in Figure 4-5.

Figure 4-2. Canada's MBT, the Leopard 2A6M.

Canada’s MBT, the Leopard 2 weighs 63.2 tonnes when fully loaded and is representative of an MLC 70 tracked vehicle. The Leopard 2 has a ground contact length of 5.0 m and 0.64 m wide tracks.

Figure 4-3. Heavy Logistics Vehicle Wheeled (HLVW) with a Loaded Low-Bed.

For the testing program, a low-bed trailer was attached to the HLVW. The trailer was weighed down with a 27 tonne loader. The vehicle configuration weighed 50.4 tonnes dispersed over five axles, and was representative of an MLC 49 wheeled vehicle. The HLVW and trailer was 18.7 m long, with a vehicle width of 2.8 m.

Figure 4-4. Engineer Light Armoured Vehicle (E-LAV).

The E-LAV has four axles in its short wheelbase of 4.2 m. The E-LAV has a total width of 3.4 m and a narrow wheel track of 2.8 m. The vehicle variant used in testing has a maximum weight of 27.5 tonnes, which is representative of an MLC 30 wheeled vehicle.

Figure 4-5. Cougar.

The Cougar has three axles in its wheelbase of 4.9 m. The Cougar is 2.7 m wide and has a wheel track of 2.6 m. The Cougar represents an MLC 29 wheeled vehicle, when loaded to its maximum weight of 26.1 tonnes.

4.4.2 Test Set-Up

4.4.2.1 Site and Bridge Information

The Mountbatten Bridge, shown in Figure 4-6, located at 4th Canadian Support Base (CDSB) Petawawa in Ontario, Canada was the location selected for testing. The bridge was constructed in 1977 and 1978 by the British Army Royal Engineers. The Mountbatten Bridge has two spans of 29.6 m and 32.9 m, with four continuous steel stringers joined compositely to a concrete deck. The bridge has a 6.0 m roadway width and 7.5 m overall width. A typical cross-section of a main steel girder is shown in Figure 4-7 (note the longitudinal stiffeners were only located on the outside of the exterior girders in the negative span location), and the cross-section of the entire bridge is shown in Figure 4-8.

Figure 4-6. Elevation View of the Mountbatten Bridge from the South-East Shore.

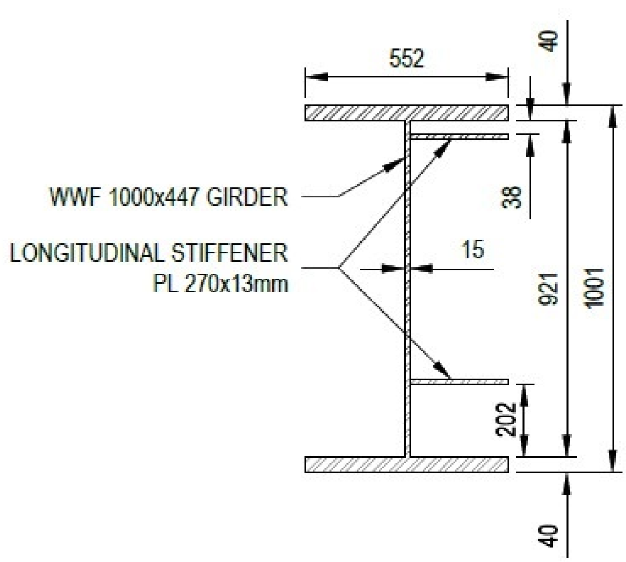

Figure 4-7. WWF 1000x447 Girder Properties (all dimensions in mm).

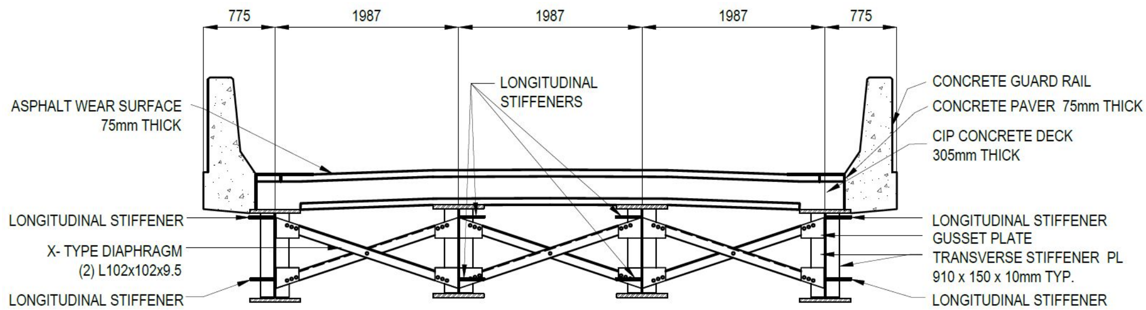

Figure 4-8. Mountbatten Bridge Cross Section (all dimensions mm).

4.4.2.2 Instrumentation

An analysis was completed to determine the preferred location for instrumentation using the commercial software CSi Bridge [18] in order to identify the maximum moment location in the longest span. The selected location was representative of the where the maximum effects generated by the vehicle loads would be observed. There were additional locations were additional instrumentation was mounted for further analyses. The data for the results presented in this paper were obtained from Location 3 (L3), which was 14.0 m from the East bearings, located on the East span shown in Figure 4-9. 10-millimeter 120-ohm strain gauges were installed on the girders at this location in a configuration shown in Figure 4-10, which were the main sources of data for this paper.

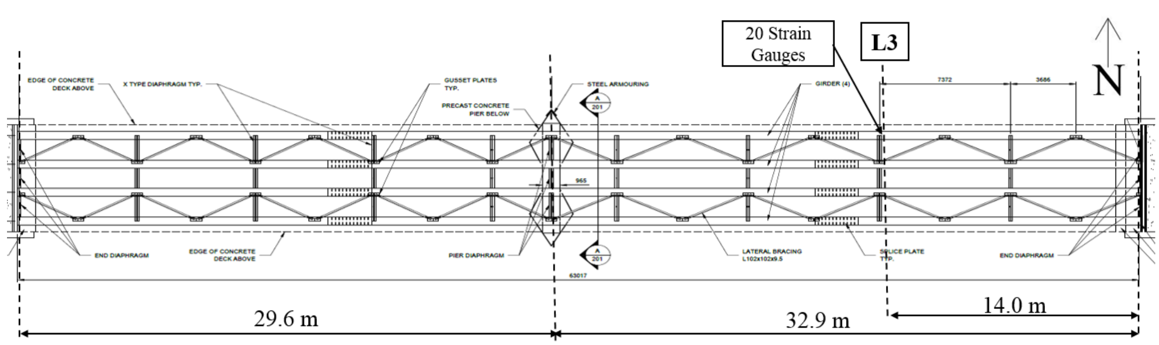

Figure 4-9. Instrumentation Locations.

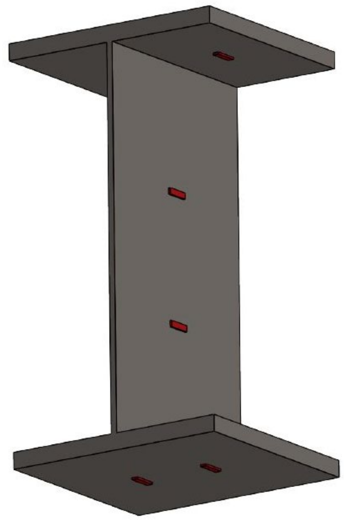

Figure 4-10. Typical Strain Gauge Installation Locations.

The data used for computing the DEF and BEF values was the maximum response observed from strain gauges installed on the main girders, which was obtained from the underside of the bottom flange. Multiple installed gauges provided a clear picture of the stress distribution with girder depth, and the two gauges on the bottom flange provided redundancy for the critical data source. The instrumentation was connected to an HBM MGCplus data acquisition system that collected data at a rate of 1200 Hertz to avoid data aliasing. Multiple data acquisition systems were used to account for the other instrumentation installed on the bridge, controlled by one computer to synchronize test results.

4.4.3 Test Program

The static response for all vehicles' were obtained along the centerline, and along the edge of the bridge for the Leopard 2. Axle positioning affected the maximum bridge response, and therefore, vehicles conducted their static response tests by travelling in both directions. In order to capture the maximum static response, all vehicles would crawl (speed less than 5 km/h) onto the bridge, stop at L3, and then crawl off the bridge. As the maximum static response may not have been obtained during the stationary portion, the crawling data was used to validate that the maximum static response was captured [9].

The Leopard 2 underwent a comprehensive test program over a series of obstacles at varying speed intervals between 10 km/h and 50 km/h. The obstacles were constructed out of nominal 2" x 10" x 12' Spruce-Pine-Fir (SPF) lumber which produced obstacles that were 38 mm tall for the single high obstacles, 76 mm tall for the double high obstacles, and 114 mm tall for the triple high obstacle. The single and double high obstacles were representative of significant obstacles while the triple high obstacles were extreme obstacles. The triple high obstacles were representative of such a surface irregularity that they would prevent some vehicles, specifically wheeled, from crossing. For both the single and double high obstacles, five obstacles were laid on the bridge roadway surface spaced at 3.0 m centre-to-centre, shown in Figure 4-11. This orientation and setup for the obstacles was selected to generate the worst possible dynamic effects for the

Leopard 2, as after the tank would crest an obstacle, the front tracks would again make contact with the roadway before cresting the subsequent obstacle, shown in Figure 4-12. The triple high obstacle was a single stack of three pieces of lumber (114 mm tall), placed inline with L3. The wheeled vehicles carried out a modified obstacle test programs that involved only going over the significant obstacle pattern at 10 km/h, 20 km/h, and 30 km/h. The obstacles were stationed around L3 as it falls on the longest span, therefore the greatest response would be observed on that span.

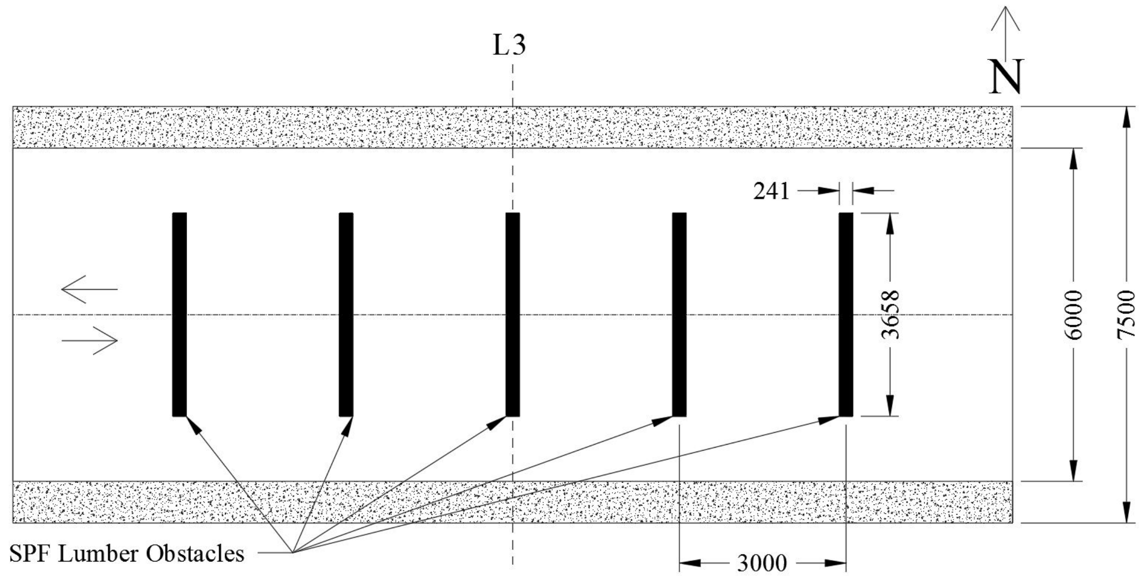

Figure 4-11. Plan-view of centerline layout for single and double high obstacles (all dimensions mm).

Figure 4-12. Leopard 2 travelling over a series of five 38 mm tall obstacles spaced 3.0 m apart.

A series brake check tests were carried out with all vehicles. The Leopard 2 carried out brake check tests at 10 km/h, 30 km/h, and 50 km/h, while the wheeled vehicles carried out brake check tests at 10 km/h and 30 km/h. All vehicles would travel at the stated test speed and then brake as rapidly as possible with the aim of stopping at the position that would generate the maximum response at L3. A summary of all tests completed is shown in Table 4-1. The tests carried out by all vehicles assist in further delineating the different dynamic effects generated by different types of vehicles, and provide insight on the effects that surface irregularities and braking can cause.

Table 4-1. Surface Irregularity and Braking Test Program.

Tests	Vehicle Speed (km/h)				
	10	*20*	*30*	*40*	*50*
Centerline with Single High (38 mm)	T	T	T	T	T
Centerline with Double High (76 mm)	T W	T W	T W	T	T
Edge with Double High (76 mm)	T	T	T	T	T
Centerline with One Triple High (114 mm)	T		T		T
Brake Check	T W		T W		T

Note: T represents tests carried out by the tracked vehicle, the Leopard 2.
W represents tests carried out by all three wheeled vehicles.

4.4.4 Procedures

All instrumentation was installed, calibrated, and verified prior to the testing. Preliminary results were used to verify the functionality and quality of data obtained from the instrumentation. The preliminary data indicated stiffer than expected behaviour, which is attributed to the fully connected concrete barriers. An image of the stress distribution, taken from preliminary data is shown in Figure 4-13. The linear distribution of stress with depth is evident which contributes to the reliability of the data obtained from the girder strain gauges. Prior to each test, all instrumentation was zeroed and a new data capture sequence commenced in order to eliminate any thermal effects on the strain measurements throughout the testing program.

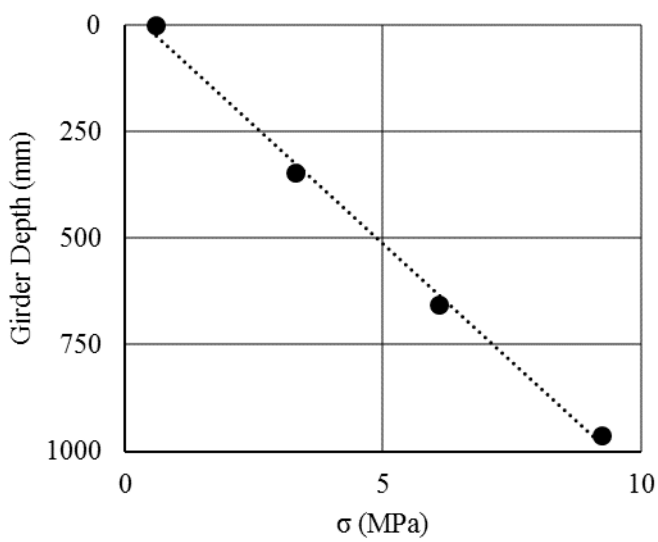

Figure 4-13. Typical Stress (σ) Distribution with Girder Depth when Subjected to a Vehicle Load.

The static response of all vehicles were critical tests and the results of those tests were incorporated into the computation of all DEF and BEF values. Therefore, the accuracy of DEF and BEF results relied on obtaining quality stationary results. The static response for all bottom flange strain gauges, for all vehicles travelling in both directions, were graphed in order to review the validity of the maximum response observed. The maximum observed static response was affected by the braking and acceleration of the test vehicles and it was necessary to appropriately account for this in the determination of the static response. Similar results were observed in the conduct of the braking tests where the braking occurred over as short a distance as possible. The effects from braking and accelerating are demonstrated in Figure 4-14, which is data from a strain gauge located on the bottom of an interior girder.

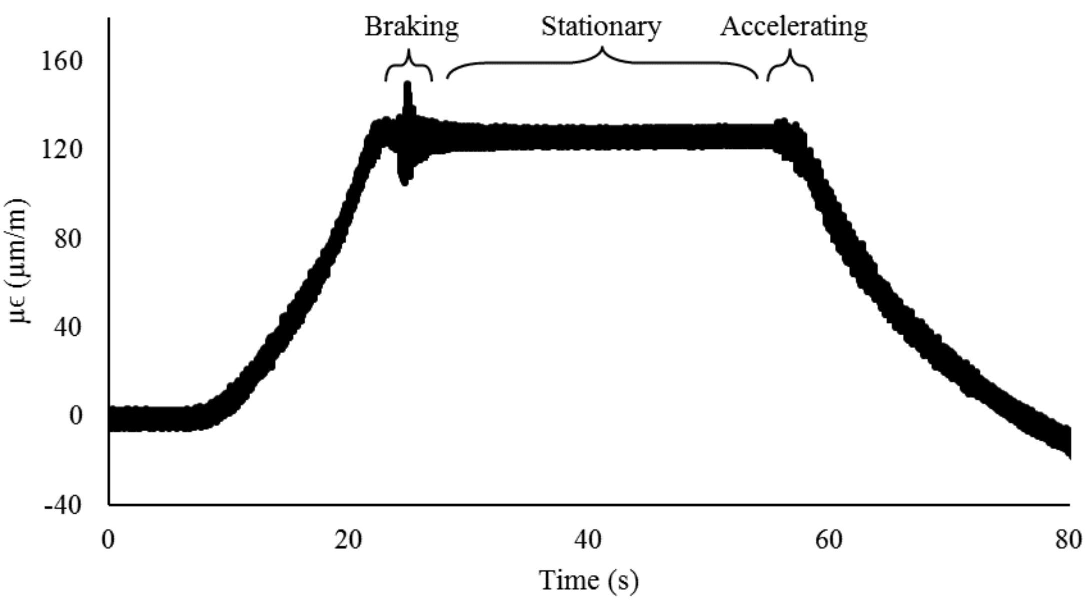

Figure 4-14. Interior Girder Static Response for the Leopard 2 at L3.

The results from all tests were compiled for review. With the maximum static responses' validated, they were then compared to the maximum dynamic responses for each test and for each vehicle. The maximum observed dynamic response was compared to the maximum observed static response using Equation 4-1 or Equation 4-2 depending on the test performed, using the procedure outlined in Section 4.2.6. The obstacles and braking tests were centered over L3 as that was representative of the longest span and where the maximum values would be observed. DEF and BEF values obtained from L3 compared the results from eight strain gauges.

4.5 Results

When obstacles were placed on the bridge deck surface, they generated marked changes in the dynamic behaviour of all test vehicles. The wheeled vehicles showed a notable and sporadic increase in dynamic behaviour while the Leopard 2 dynamic responses were relatively minimal compared to those of the wheeled vehicles. The Leopard 2 capabilities permitted it to carry out a more comprehensive test program over more varied obstacles than the wheeled vehicles.

4.5.1 Obstacles – Leopard 2

The Leopard 2 generated markedly higher DEF values travelling over obstacles compared to the results of it travelling over a smooth surface (discussed in Chapter 3). DEF results for all of the obstacle tests completed by the Leopard 2 are shown in Figure 4-15. As anticipated, the more severe obstacles generated higher and more variable DEF values. The results do highlight how significant surface roughness, or obstacles, will increase the dynamic loading effects of vehicles. The single and double high obstacles were repeated five times, which caused repeated oscillations and dynamic impact from the Leopard 2. While the triple high obstacle would represent extreme damage, it would be small when compared to deliberately placed objects, such as logs placed across a bridge as obstacles. If a bridge would be near load capacity when being traversed by a specific heavy vehicle, consideration should be given to the size of damage or obstacles before vehicles are permitted to cross. The Leopard 2 crew had no concerns about travelling over the triple high obstacle, which reinforces the capability of tracked fighting vehicles and indicates that perhaps vehicle operators may not fully grasp the loading effects that they can cause to a bridge. The largest single stack of lumber was only 114 mm tall, and still produced a maximum dynamic amplification of 1.54. There does not seem to be a clear relationship between vehicle speed and the dynamic effects generated. However, one can extrapolate that if obstacles larger than 254 mm (still traversable by a tank) were ever used on a bridge, the dynamic effects generated by a tank crossing them may be sufficiently large to potentially overload the bridge in a situation where the structure had little extra static capacity.

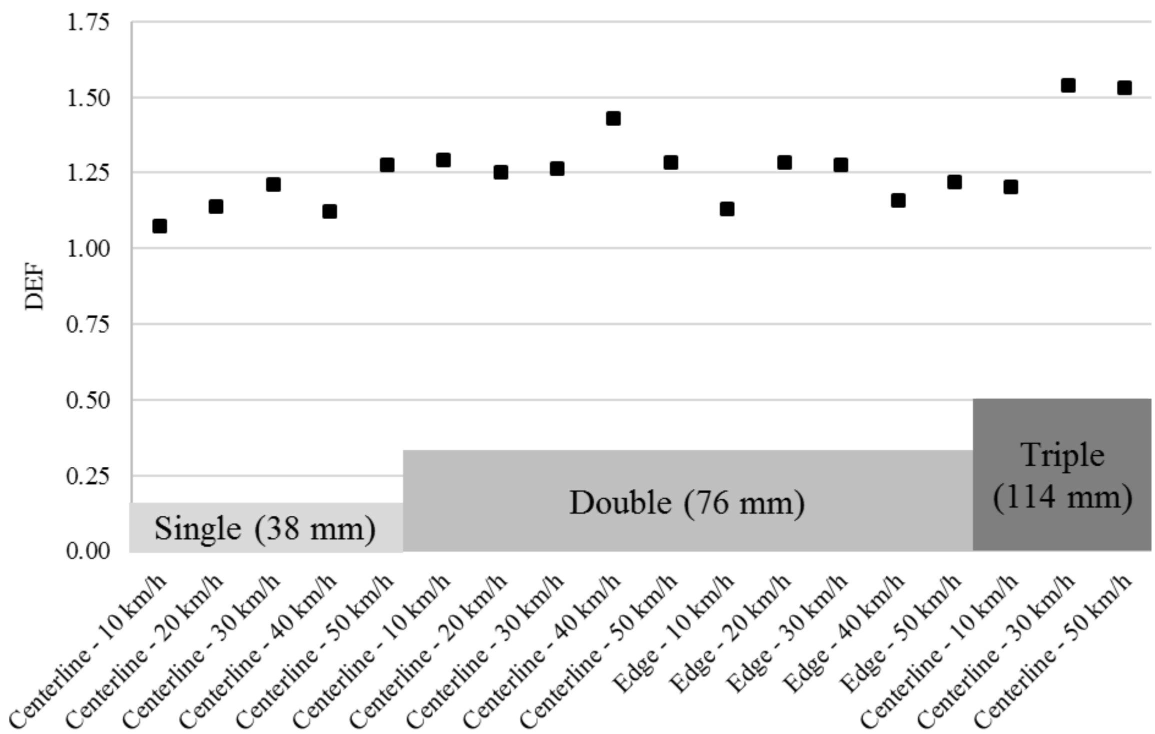

Figure 4-15. Leopard 2 Obstacle DEF Values.

DEF results from the Leopard 2 obstacle tests were grouped together in the three categories of obstacles that it crossed, and their mean and Standard Deviation (σ) values are shown in Table 4-2. Although the dataset is not extensive, it does provide some indication of the dynamic effects that obstacles of varying degree can cause.

Table 4-2. Leopard 2 Obstacle Mean DEF and Standard Deviation Values.

Obstacle Category	Mean DEF	DEF σ	Data Points
Single High (38 mm)	1.16	0.07	5
Double High (76 mm)	1.26	0.08	10
Triple High (114 mm)	1.42	0.16	3

The Leopard 2 demonstrated a Mean DEF of 1.01 when travelling over a smooth surface as presented in Chapter 3. The results from Chapter 3 and Table 4-2 were graphed with the obstacle height, as shown in Figure 4-16, in order to analyze any trends. The error bars are representative of one standard deviation.

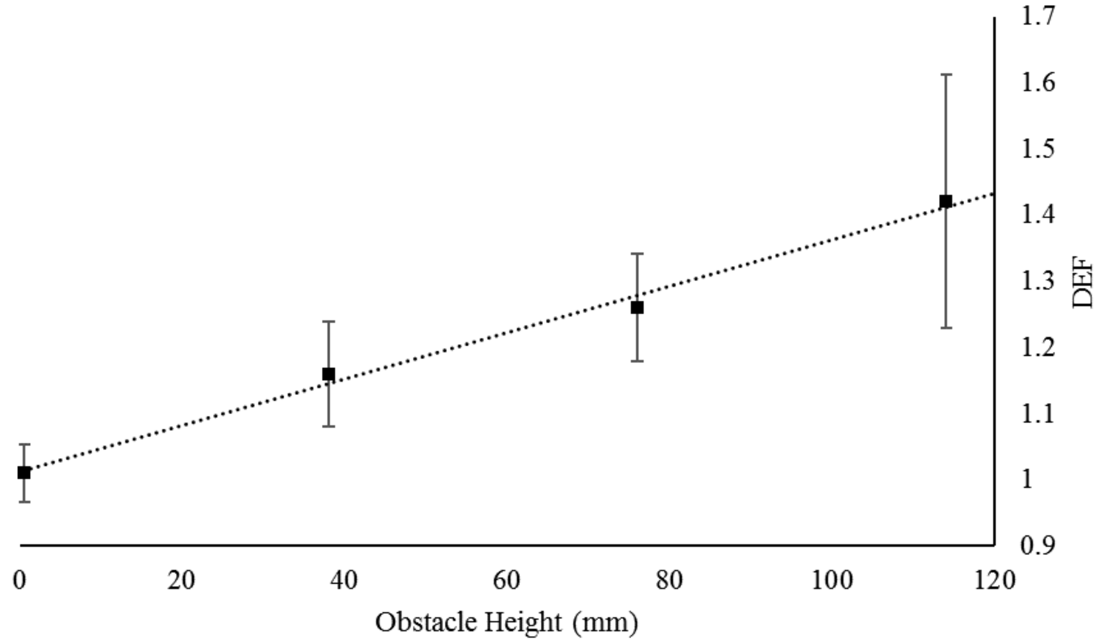

Figure 4-16. Leopard 2 DEF Trend with Obstacle Height.

A linear trend is evident in the dynamic effects generated by the Leopard 2 travelling over obstacles of varying height. The results indicate low variability in no obstacle or small obstacle situations; however, increased variability with augmented obstacle height. The linear trend can be represented by the following equation:

$$DEF_{Obstacle} = 0.0035(h) + 1.01 \qquad 4\text{-}3$$

where $DEF_{Obstacle}$ represents an estimated DEF value that could be expected when travelling over an obstacle of height (h) in millimeters. Equation 4-3 may be appropriate for use when estimating the dynamic effects generated by obstacles of varying height to be traversed by tracked vehicles, further testing is recommended to confirm this trend. If the capacity of the bridge is a concern, a Caution or Risk Crossing should be considered to mitigate the potential risk to the structure. Operators should limit their crossing speed, with the aim of minimizing the dynamic loading effects. It is likely that the DEF values for obstacle crossing at low speeds could be limited to 1.25.

4.5.2 Obstacles – All Vehicles

The wheeled vehicles only carried out obstacle tests over double high obstacles, and were only able to achieve speeds up to 30 km/h. The wheeled vehicles were unable to achieve as high of speed as the Leopard 2 due to vehicle operators' comfort levels, and they would never cross obstacles of that magnitude at high speeds. Site and vehicle limitations prevented the wheeled vehicles from carrying out tests over single high obstacles. A graphical representation of the DEF values obtained from all vehicles when travelling over similar obstacles is shown in Figure 4-17.

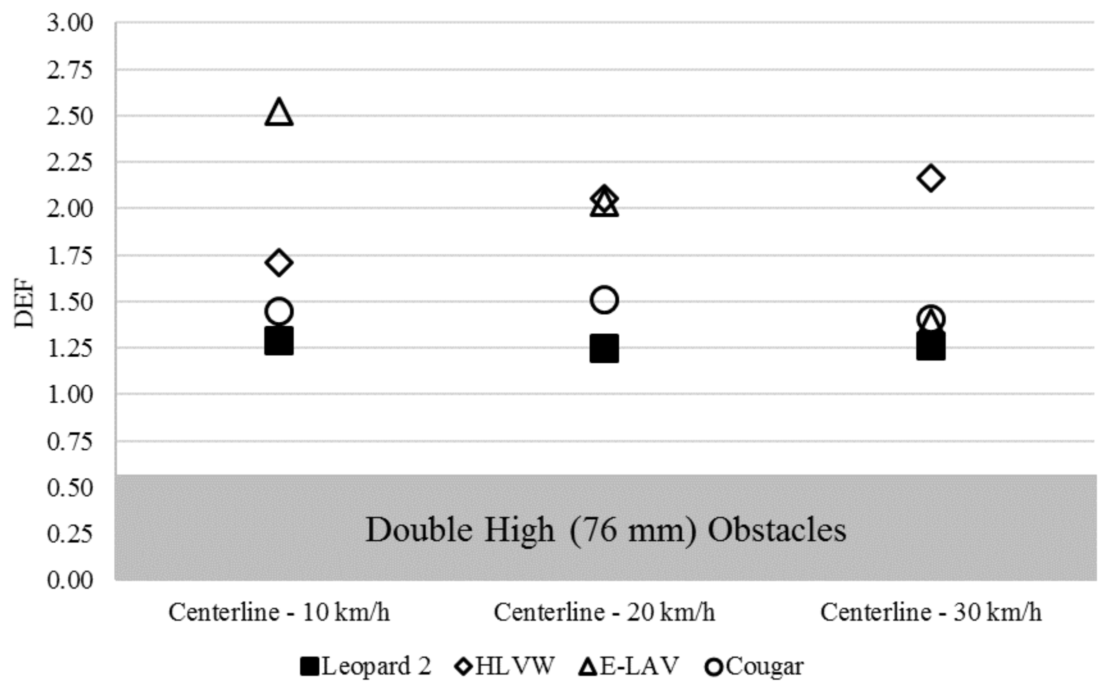

Figure 4-17. All Vehicles Similar Obstacles DEF Values.

The comparable data when all vehicles went over similar obstacles again illustrates the distinct difference in dynamic behaviour between military tracked and wheeled vehicles. Surface irregularities cause greater dynamic effects for wheeled than tracked vehicles; approximately five times in some instances. DLA values assigned in design codes are generally as a result of smooth surface tests; however, the results from obstacle tests assist in illustrating the different dynamic behaviour of different vehicle types.

Anecdotal evidence as to how varying degrees of damage/obstacles/debris can influence the dynamic effects of vehicles is important for engineers to consider. Military forces may encounter bridges that are damaged, or with deliberate obstacles placed across them. As bridges represent key strategic objectives, route denial with deliberate obstacles (e.g. trees or logs) is always possible; therefore, a risk exists of overloading when military vehicles cross bridges with obstacles. Similarly, post-natural disaster situations could involve damaged bridges or bridges covered with debris requiring use by the general population and first responders. Local engineer authorities should take into consideration the effects that any damage or debris would have on the dynamic effects generated by vehicles crossing a bridge.

Table 4-3 provides the mean DEF and Standard Deviation values for all vehicles travelling over double high (76 mm) obstacles. The Cougar was notably consistent in the dynamic effects it generated when travelling over obstacles and was generally less severe than the other wheeled vehicles.

Table 4-3. All Vehicles Double High (76 mm) Obstacle Mean DEF and Standard Deviation Values.

Vehicle	Mean DEF	DEF σ	Data Points
Leopard 2	1.26	0.08	10
HLVW	1.98	0.20	3
E-LAV	1.98	0.47	3
Cougar	1.45	0.04	3
All Wheeled Vehicles	1.80	0.38	9

Engineers should be cognisant that in situations where bridges have significant surface damage or deliberate obstacles across the bridge path, if military vehicle are to press through/over the obstacles, higher DLA values should be used in determining the capacity of that bridge. Granted, that in an operational context, time may not permit consideration of the increased load effects that obstacles will cause, however the increased associated risk in crossing should be identified. On exercises, appropriate and deliberate consideration should be given to the increased dynamic effects of any activities that see deliberate obstacles placed across bridges.

4.5.3 Braking Force

While the effects from braking force are considered in the bridge's longitudinal direction, a vertical force component with an increase in flexural stress as discussed in Section 4.2.5 is expected when vehicles brake on a bridge. The increase in strain caused due to the test vehicles braking are shown in Figure 4-18. The increase in strain as a result of braking, is represented by the term BEF and was calculated using Equation 4-2 as described in Section 4.2.5. The Leopard 2 was the only vehicle that conducted a brake check test at 50 km/h.

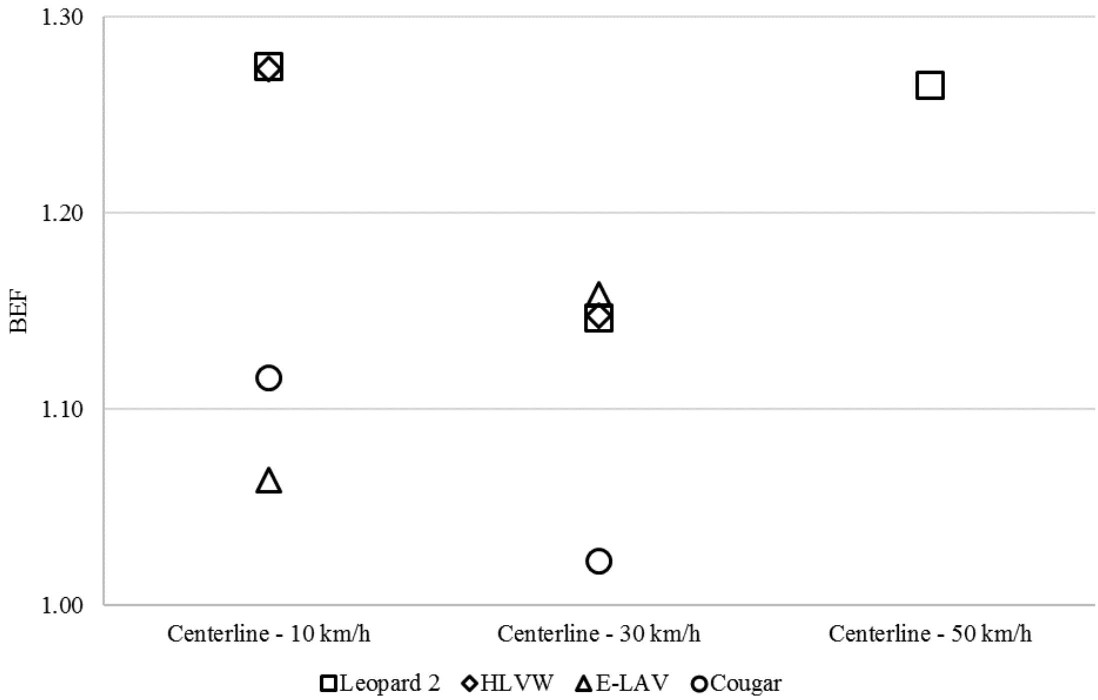

Figure 4-18. All Vehicles BEF Values.

A key characteristic that was noted during the experimentation was that the Leopard 2 was able to lock-up its tracks and come to a complete stop significantly sooner and more abruptly than any of the wheeled vehicles. The HLVW brake test resulted in BEF values nearly identical to those of the Leopard 2, although it required a significantly greater distance to come to a complete stop, the interaction between the tractor and trailer components of the HLVW caused the increased strain response. Analysis in Chapter 3 showed that the Leopard 2 consistently produced lower DEF/DLA values than the wheeled vehicles, yet in braking the Leopard 2 was consistently at the top boundary of results, as a result of its ability to stop in an extremely short distance. The Leopard 2 was the heaviest tested vehicle and it demonstrated the highest BEF value of a 27% increase in its load effects.

While braking effects are not considered within the *CSA S6-14 CHBDC* provisions for dynamic load allowance, these results indicate how at low speeds and with a short stopping distance, both the Leopard 2 and the HLVW (similar to a civilian tractor-trailer) can produce dynamic effects greater than the design code DLA value of 0.25. While the *CSA S6-14 CHBDC Commentary* does attribute braking to be a rare event likely to not occur by overloaded vehicles, the test results are nonetheless noteworthy because they indicate how a short braking distance (e.g. in an emergency) could exceed DLA values. Specific to a military context, tank operators should be aware of the increased load effects that they can generate by rapidly stopping on a bridge.

4.6 Conclusions

The experimental program presented in this paper was carried out to further examine the difference between the dynamic behaviour between wheeled and tracked military vehicles, to examine how vehicles' dynamic effects are increased by surface irregularities or obstacles, and to examine the effects of braking on the dynamic effects generated by vehicles. Design codes do not appear to have provisions for increasing the DLA when bridge surfaces are no longer smooth. In bridge design, the braking force of vehicles is treated as a longitudinal force applied at the deck, while in reality there is also an increase in the flexural forces. A summary of the key results and observations are presented below:

1. Tracked and wheeled vehicles exhibit distinctly different dynamic behaviour, which is increasingly apparent when travelling over surface irregularities/obstacles.

2. Operators of tracked vehicles should be cognisant of the dynamic effects that they can generate when traversing obstacles. A small tree or other obstacle placed across a bridge could easily increase the dynamic loading effects by over 50%. Depending on the load capacity rating of the bridge, a significant risk of overloading could exist.

3. In some instances, the dynamic effects generated by wheeled vehicles can be approximately five times (500%) that of a tank.

4. Situations (e.g. natural disasters) that see bridges with damage or debris, carry the risk of increased dynamic effects that may risk overloading a bridge.

5. The dynamic effects generated by vehicles when braking are significantly increased as stopping distance is reduced.

6. The Leopard 2, the tested tracked vehicle and Canada's main battle tank, is capable of stopping within an extremely short distance, which if done on a bridge can cause a dynamic loading increase greater than both North American Military and Canadian Design Code provisions for DLA.

7. Military doctrine should be amended to emphasize that even under Normal Crossing Conditions, the rapid braking of vehicles should be avoided on bridges.

8. Further research into the dynamic effects generated by alternate surface irregularities/obstacles should be carried out. This research should include surface obstacles representative of potholes or gaps in the roadway surface.

9. The results presented on the braking effects of vehicles may be anecdotal; however, they do demonstrate an increase in the dynamic effects generated by braking which may require further testing and analysis.

4.7 Acknowledgements

This research was funded by the Department of National Defence and would not have been possible without the support and cooperation of several Canadian Armed Forces directorates and units, specifically 2 Combat Engineer Regiment, Director Combat Support Equipment Management, and Director Armament Sustainment Program Management.

4.8 References

[1] *NATO Standardization Agreement 2021 - Military Load Classification of Bridges, Ferries, Rafts and Vehicles (Edition 8),* NATO, 2015.

[2] A. MacDonald, "Applying Probabilistic Methods to the NATO Military Load Classification System for Bridges," Western University Master's Thesis, London, 2014.

[3] DND/MDN Canada, *B-GL-361-014FP-001 MANUAL FOR MILITARY NONSTANDARD FIXED BRIDGES (DRAFT),* Ottawa, 2008.

[4] CSA Group, *Commentary on CSA S6-14, Canadian Highway Bridge Design Code,* Mississauga: CSA Group, 2014.

[5] R. Lenner, M. Keuser and M. Sykora, "Assessment of existing reinforced concrete bridges exposed to military loads," in *Novák and Vořechovský: Proceedings of the 11th International Probabilistic Workshop*, Brno, 2013.

[6] Department of the Army, *FM 3-43.343 Military Nonstandard Fixed Bridging,* Washington, 2002.

[7] CSA Group, *S6-14, Canadian Highway Bridge Design Code,* Mississauga, Ontario: CSA Group, 2014.

[8] L. Deng, Y. Yu, Q. Zou and C.S. Cai, "State-of-the-Art Review of Dynamic Impact Factors of Highway Bridges," *Journal of Bridge Engineering,* vol. 20, no. 5, pp. 04014080-1 to 04014080-14, 2015.

[9] P. Paultre, J. Proulx and M. Talbot, "Dynamic Testing Procedures for Highway Bridges Using Traffic Loads," *Journal of Structural Engineering,* vol. 121, no. 2, pp. 362-376, 1995.

[10] B. Bakht and S.G. Pinjarkar, *Dynamic Testing of Highway Bridges - A Review,* Washington: Transportation Research Record 1223, Transportation Research Board, 1989, pp. 93-100.

[11] R. Shepherd and R.J. Aves, "Impact Factors for Simple Concrete Bridges," in *Proc. Inst. Civ. Eng., Vol. 55 Part 2, Research and Theory, Paper 7548*, 1973.

[12] J. Billing, "Dynamic Loading and Testing of Bridges in Ontario," *Canadian Journal of Civil Engineering,* vol. 11, no. 4, pp. 833-843, 1984.

[13] J. Page, "Dynamic Wheel Load Measurements on Motorway Bridges," Department of Environment, Transport and Road Research Laboratory, TRRL Report LR 722, Crowthorne, 1976.

[14] I. Paeglite and A. Paeglitis, "The Dynamic Amplification Factor of the Bridges in Latvia," in *11th International Conference on Modern Building Materials, Structures and Techniques, MBMST 2013*, Vilnius, 2013.

[15] Department of the Army, *FM 5-102 Countermobility,* Washington, 1985.

[16] S.S. Law and X.Q. Zhu, "Bridge Dynamic Response due to Road Surface Roughness and Braking of Vehicle," *Journal of Sound and Vibration,* vol. 282, pp. 805-830, 2005.

[17] R. Cantieni, "Dynamic Load Tests on Highway Bridges in Switzerland: 60 Years Experience of EMPA," Swiss Federal Laboratories for Materials and Testing Research, Dubendorf, 1983.

[18] Computers and Structures, Inc., *CSi Bridge,* Walnut Creek, CA: v20.1.0, 2018.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 General

This research focussed on validating the different dynamic behaviour and loading effects between wheeled and tracked vehicles. The consequences of surface irregularities and braking on the dynamic loading effects was also examined in order to further differentiate between the dynamic loading effects of wheeled and tracked vehicles.

Extensive field testing was conducted on a two-span concrete steel girder composite bridge, resulting in a total of 90 load tests, and each test providing 95 channels of data sampled at 1200 Hz fully representing the behaviour of the bridge. The test program involved: static load tests for all vehicles, tests over a smooth surface both along the centerline and the edge of the bridge, tests over a series of obstacles to simulate surface irregularities, and a series of brake check tests.

The tracked fighting vehicle was able to complete a more comprehensive test program than the wheeled vehicles. The culmination of this project was to provide a quantifiable difference in the dynamic behaviour of wheeled and tracked vehicles in order to suggest a reduced DLA for tracked vehicles. A reduced DLA for tracked vehicles would result in a marked increase in the mobility of military tracked fighting vehicles (e.g. tanks) both at home on exercises and abroad on operations. Increased bridge MLC capacity would increase the lifespan of existing bridge infrastructure that is currently trafficked by military vehicles because they would not imminently require upgrading to accommodate increasing tracked fighting vehicle mass.

5.2 Conclusions

All vehicles performed a similar test program for the smooth surface tests. Results indicate very similar mean DEF values, all close to 1.00. Each vehicle demonstrated individual degrees of variability in its results. The variability in DEF results is what produces higher DLA values. As anticipated, the Leopard 2 was very consistent in the dynamic effects that it generated. The wheeled vehicles were all variable in their observed DEF values which contributed to their individually and grouped higher calculated DLA values.

Surface irregularities were simulated by placing different combinations and quantities of lumber planks across the bridge surface. The addition of obstacles created marked dynamic increase for all vehicles. The Leopard 2's capabilities permitted it to complete tests over varied obstacles of increased size. When examined in isolation, the Leopard 2's increase in dynamic effects as a result of obstacles appears quite large. However, when the similar tests completed by wheeled vehicles are examined, the distinct difference in dynamic behaviour between wheeled and tracked vehicles is further evident. Wheeled vehicles produced dynamic effects approximately five times that of the tracked tank in some obstacle situations. Code provisions for DLA are generally focussed on smooth surfaces, yet there remains a possibility for obstacles or surface irregularities to be present in military situations as a result of deliberate obstacles or damage. Anecdotal results as to the degree in which even small surface irregularities can increase dynamic effects is important for all military vehicle operators to understand.

Braking effects are generally applied in the longitudinal direction in design code provisions and separate from DLA. Due to the shift in the vehicle load during braking, one would expect there to be an increase in vertical loads and corresponding flexural forces as a result of braking. The effects of braking were examined and when wheeled vehicles attempted to stop over as short of a distance of possible, they generated large dynamic effects. Some wheeled vehicles remained within design code provisions for DLA while the HLVW generated dynamic effects greater than Canadian Design Code provisions for DLA. The Leopard 2 was able to stop in a shorter distance, which in some instances created dynamic effects greater than Canadian Design Code provisions for DLA. The results demonstrate how even at low speeds and at short stopping distances, large dynamic effects can be generated.

A summary of the key results and observations from each manuscript is presented below:

1. Tracked and wheeled vehicles exhibit distinctly different dynamic behaviour. This is increasingly evident when vehicles travel over surface irregularities.

2. Out of the wheeled vehicles, the E-LAV test data consistently results in higher DLA values due to its variable dynamic behaviour.

3. The STANAG 2021 recommended reliability index and live-load factor values result in a calculated DLA for wheeled vehicles of 0.12. The calculated DLA value is less than the current recommended DLA of 0.15 for military traffic. Therefore, these results are supportive of the reduced DLA value, of 0.15, for military traffic.

4. The Leopard 2 DLA values are consistently less than two-thirds of the DLA values for all wheeled vehicles. As such, it may be appropriate to reduce design code provisions of DLA values used for wheeled traffic by one-third for tracked vehicles, which would represent a significant strategic increase in the mobility of tracked vehicles.

5. Application of a reduced DLA for tracked vehicles could result in an increase to existing bridge capacity of 5% to 13% for tracked vehicles.

6. Observed and calculated DLA values for the Leopard 2 and All Wheeled Vehicles as a result of experimentation fell within all examined design code DLA provisions.

7. Tracked vehicles (e.g. tanks) are able to easily pass over significant obstacles and operators of tracked vehicles should be cognisant of the dynamic loading effects that they can generate when traversing obstacles. A small tree or other obstacle placed across a bridge could easily increase the dynamic effects by over 50%. Depending on the load capacity rating of the bridge, a significant risk of overloading could exist.

8. In some instances, the dynamic effects generated by wheeled vehicles crossing over a rough surface or obstacle can be approximately five times that of a tank.

9. The dynamic effects generated by vehicles when braking are significantly increased as stopping distance is reduced. The Leopard 2 is capable of stopping within an extremely short distance, which if done on a bridge can cause a dynamic increase greater than both North American Military and Canadian Design Code provisions for DLA.

5.3 Recommendations

Based on the scope of the research carried out, recommendations are provided for future load testing of bridges. These recommendations are particularly related to the testing of military vehicles for further data on smooth surface dynamic effects, for military vehicles travelling over obstacles, and for the braking effects of vehicles.

1. More research between the dynamic effects of tracked versus wheeled vehicles should be carried out by varied tracked vehicles and on different bridge types, in order to further support the use of a reduced DLA for tracked vehicles.

2. Further research should be conducted on the combined effects of live-load and DLA when analyzing the effects of military wheeled and tracked vehicles on bridges. It is anticipated that a combined reduction would significantly increase the mobility of military forces.

3. Military doctrine and MLC software tools should be amended to take into consideration a reduced DLA for tracked military vehicles, equal to two-thirds of that used for wheeled military vehicles.

4. Military doctrine should be amended to emphasize that even under Normal Crossing Conditions, the rapid braking of vehicles should be avoided on bridges.

5. Further research into the dynamic effects generated by alternate surface irregularities/obstacles should be carried out. This research should include surface obstacles representative of potholes or gaps in the roadway surface.

6. The results presented on the braking effects of vehicles may be anecdotal; however, they do demonstrate an increase in the dynamic effects generated by braking. Further research into the dynamic effects generated by short distance braking should be carried out.

REFERENCES

[1] Oxford English Dictionary, Oxford University Press, 2004.

[2] Statistics Canada, "Canada's Core Public Infrastructure Survey: Roads, bridges and tunnels, 2016," 2018.

[3] CSA Group, *S6-14, Canadian Highway Bridge Design Code,* Mississauga, Ontario: CSA Group, 2014.

[4] *NATO Standardization Agreement 2021 - Military Load Classification of Bridges, Ferries, Rafts and Vehicles (Edition 8),* NATO, 2015.

[5] S. Penstone, R. Tanovic, J. O'Reilly and G. Wight, *CABMLC Release 1.0 (Internal Version 2.23.1),* Kingston: DND/MDN Canada. RMC-CMR Department of Civil Engineering., 2019.

[6] S. Penstone, R. Tanovic, J. O'Reilly and G. Wight, *RFBMLC V2.3,* Kingston: DND/MDN Canada. RMC-CMR Department of Civil Engineering., 2019.

[7] S. Penstone, R. Tanovic, J. O'Reilly and G. Wight, *CORVMLC v1.3,* Kingston: DND/MDN Canada. RMC-CMR Department of Civil Engineering., 2019.

[8] S. Penstone, R. Tanovic, J. O'Reilly and G. Wight, *CAVMLC v2.9.6,* Kingston: DND/MDN Canada. RMC-CMR Department of Civil Engineering., 2015.

[9] Royal Military College of Canada, "Thesis Preparation Guidelines," Royal Military College of Canada, Kingston, 2015.

[10] A.J. MacDonald, "Applying Probabilistic Methods to the NATO Military Load Classification System for Bridges," Western University Master's Thesis, London, 2014.

[11] DND/MDN Canada, *B-GL-361-014FP-001 MANUAL FOR MILITARY NONSTANDARD FIXED BRIDGES (DRAFT),* Ottawa, 2008.

[12] University of Munich, *BRASSCO-NG,* Munich, 2019.

[13] CSA Group, *Commentary on CSA S6-14, Canadian Highway Bridge Design Code,* Mississauga: CSA Group, 2014.

[14] R. Lenner, M. Keuser and M. Sykora, "Assessment of existing reinforced concrete bridges exposed to military loads," in *Novák and Vořechovský: Proceedings of the 11th International Probabilistic Workshop*, Brno, 2013.

[15] R. Lenner, "Safety Concept and Partial Factors for Military Assessment of Existing Concrete Bridges," Universität Der Bundeswehr München, Munich, 2014.

[16] Department of the Army, *FM 3-43.343 Military Nonstandard Fixed Bridging,* Washington, 2002.

[17] L. Deng, Y. Yu, Q. Zou and C.S. Cai, "State-of-the-Art Review of Dynamic Impact Factors of Highway Bridges," *Journal of Bridge Engineering,* vol. 20, no. 5, pp. 04014080-1 to 04014080-14, 2015.

[18] P. Paultre, J. Proulx and M. Talbot, "Dynamic Testing Procedures for Highway Bridges Using Traffic Loads," *Journal of Structural Engineering,* vol. 121, no. 2, pp. 362-376, 1995.

[19] B. Bakht and S.G. Pinjarkar, *Dynamic Testing of Highway Bridges - A Review,* Washington: Transportation Research Record 1223, Transportation Research Board, 1989, pp. 93-100.

[20] R. Shepherd and R.J. Aves, "Impact Factors for Simple Concrete Bridges," in *Proc. Inst. Civ. Eng., Vol. 55 Part 2, Research and Theory, Paper 7548*, 1973.

[21] J.R. Billing, "Dynamic Loading and Testing of Bridges in Ontario," *Canadian Journal of Civil Engineering,* vol. 11, no. 4, pp. 833-843, 1984.

[22] M.F. Green and N. Cebon, "Dynamic Response of Highway Bridges to Heavy Vehicle Loads: Theory and Experimental Validation," *Journal of Sound and Vibration,* vol. 170, no. 1, pp. 51-78, 1994.

[23] AASHTO, "Standard Specifications for Highway Bridges," Washington, 1992.

[24] AASHTO, "LRFD Bridge Design Specifications," Washington, 2012.

[25] Ministry of Transport of the People's Republic of China (MTPRC), "General Code for Design of Highway Bridges and Culverts," Beijing, 1989.

[26] Ministry of Transport of the People's Republic of China (MTPRC), "General Code for Design of Highway Bridges and Culverts," Beijing, 2004.

[27] New Zealand Transport Agency (NZTA), "Bridge Manual," Wellington, 2013.

[28] European Committee for Standardization (CEN), "Eurocode 1: Actions on Structures—Part 2: Traffic Loads on Bridges," Brussels, 2003.

[29] British Standards Institution (BSI), "Steel, Concrete and Composite Bridges. Part 2: Specification for Loads.," London, 2006.

[30] Japan Road Association (JRA), "Specifications for Highway Bridges. Part 1: Common Specifications," Tokyo, 1996.

[31] R. Cantieni, "Dynamic Load Tests on Highway Bridges in Switzerland: 60 Years Experience of EMPA," Swiss Federal Laboratories for Materials and Testing Research, Dubendorf, 1983.

[32] J. Page, "Dynamic Wheel Load Measurements on Motorway Bridges," Department of Environment, Transport and Road Research Laboratory, TRRL Report LR 722, Crowthorne, 1976.

[33] I. Paeglite and A. Paeglitis, "The Dynamic Amplification Factor of the Bridges in Latvia," in *11th International Conference on Modern Building Materials, Structures and Techniques, MBMST 2013*, Vilnius, 2013.

[34] S.S. Law and X.Q. Zhu, "Bridge Dynamic Response due to Road Surface Roughness and Braking of Vehicle," *Journal of Sound and Vibration,* vol. 282, pp. 805-830, 2005.

[35] DND/MDN Canada, *A6M Technical Data - Technical Manual 2350/089-10,* Ottawa, 2007.

[36] DND/MDN Canada, *C-30-B40-000/MA-000 - Data Summary - Truck, Tractor, 22 Tonnes, 6x6, Super HLVW, Model H829, w/rear self loading winch,* Ottawa, 2017.

[37] DND/MDN Canada, *C-30-590-000/MB-001 - Data Summary - LAV,* Ottawa, 2016.

[38] DND/MDN Canada, *C-30-B63-000/MA-001 - Data Summary - Cougar, A2, Explosive Ordinance Disposal (EOD), ECC 160401,* Ottawa, 2018.

APPENDICES

Appendix A Instrumentation Details

1. Overview

The research presented in Chapters 2 and 3 of this thesis are a result of only a small portion of data obtained from the instrumentation. There are more aspects from the field-testing that will be explored in order to produce more Journal articles, and which may be used in additional and future research. A significant amount of instrumentation was installed on the Mountbatten Bridge to maximize the obtained data from field-testing of a bridge with several military vehicles. This Appendix outlines the process used to determine the ideal instrumentation locations, details of all the instrumentation locations, and the numbering nomenclature used.

1.1 CSi Bridge Model

A preliminary model of the Mountbatten Bridge was created in the commercial software CSi Bridge, shown in Figure A - 1. A moving load and multi-step static analysis were completed with both the CL-625 design truck and a modelled Leopard 2 tank in order to determine the locations of maximum moment. The location of maximum moments were then selected as the ideal locations for instrumentation as that is where the maximum strain would be observed, thus representing the maximum signal. The ideal instrumentation locations were rounded to the nearest 0.5 m and their locations confirmed on-site to avoid proximity to any diaphragms or stiffeners.

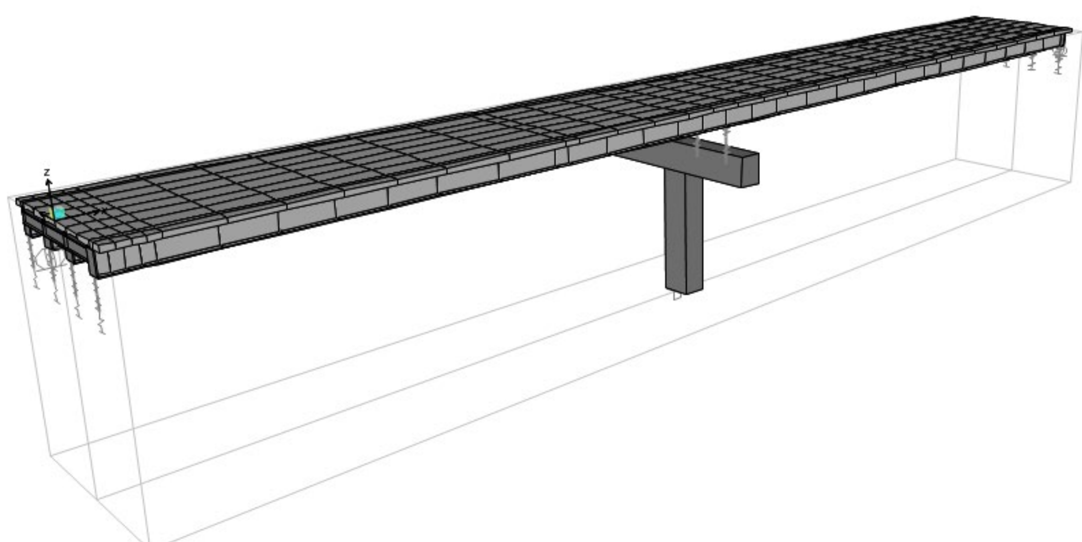

Figure A - 1. CSi Bridge Model of the Mountbatten Bridge.

The deformed shape model and a multi-step static analysis completed for the Leopard 2 are shown in Figure A - 2 which was the basis for determining the ideal primary strain gauge locations. Obtained locations were measured on site and rounded to the nearest 0.5 m to reflect the instrumentation locations as shown in Figure A - 3

Appendix A Instrumentation Details

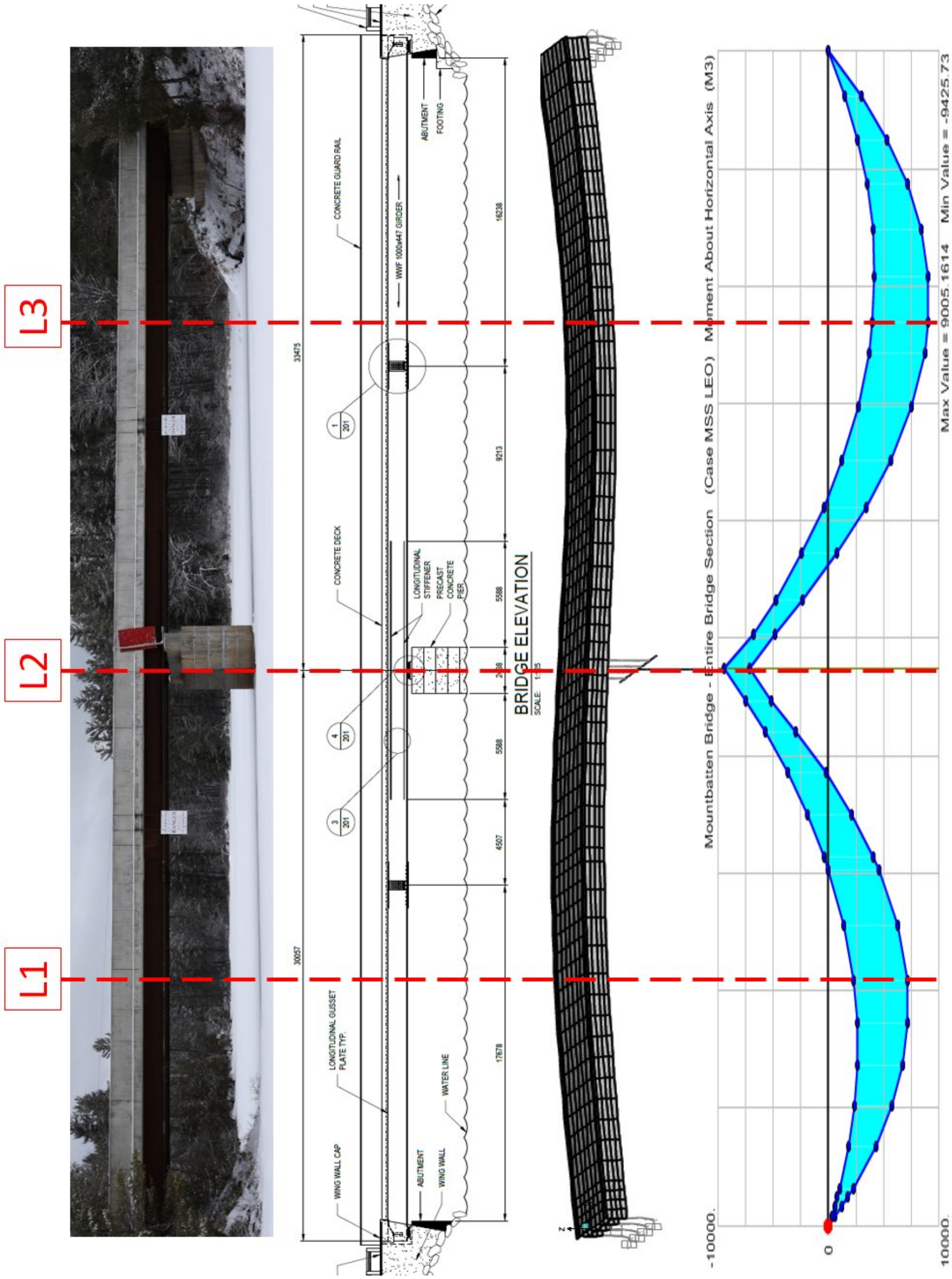

Figure A - 2. Primary Strain Gauge Locations.

Appendix A Instrumentation Details

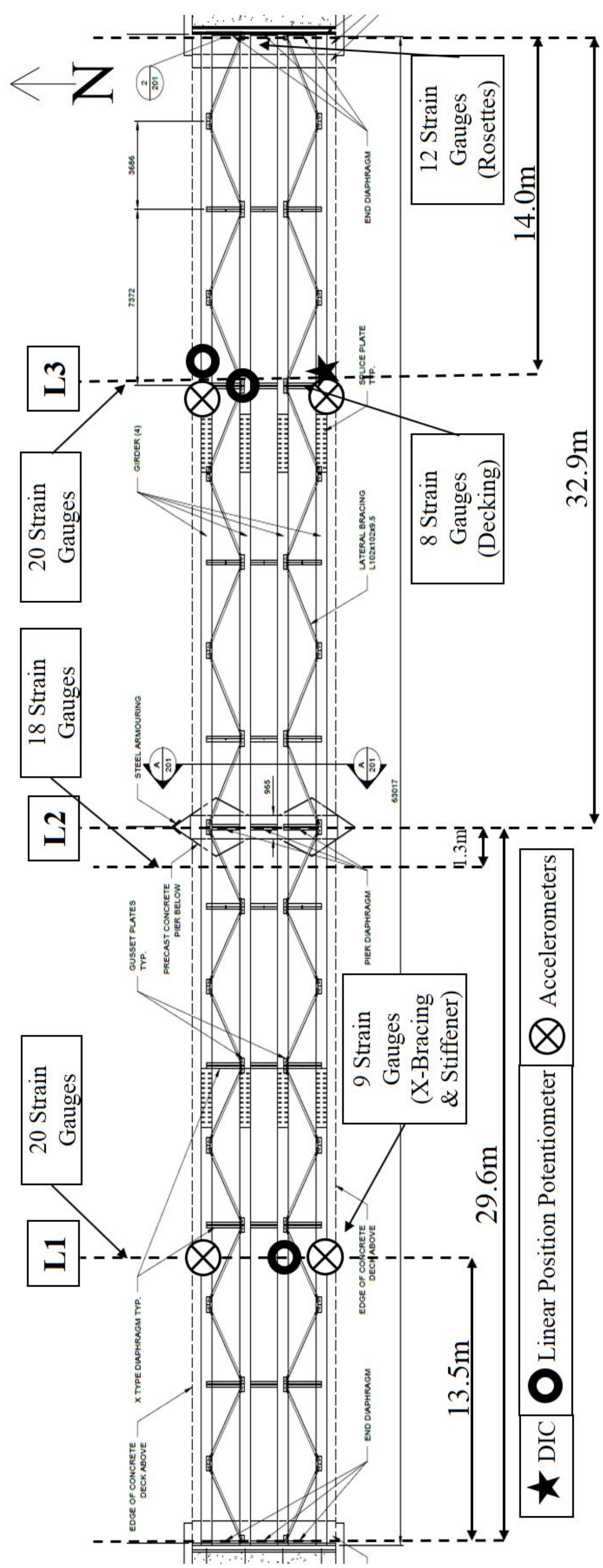

Figure A - 3. Instrumented Locations.

Appendix A Instrumentation Details

1.2 Instrumented Locations

The three primary locations of instrumentation were located in the positive moment locations and the negative moment location of the bridge. Location 1 (L1) was on the West span and at a distance of 13.5 m from the West bearings. Location 2 (L2) was located at a distance *H* (the bridge height of 1.3 m) to the West of the center pier support in the negative moment zone. Location 3 (L3) was on the East span and at a distance of 14.0 m from the East bearings. These areas were instrumented with strain gauges (on girders and steel decking), accelerometers, wired linear position string potentiometers, and a stochastic pattern for Digital Image Correlation (DIC). The DIC stochastic pattern was located on the Southern side of the exterior girder at L3.

Strain rosettes were installed at a distance *H* (the bridge height of 1.3 m) to the West of the East bearings. All four girders had rosettes installed on them. An X-type diaphragm near L1 was instrumented with a total of six strain gauges, and a lateral brace, also near L1, was instrumented with three strain gauges. The instrumented diaphragms and lateral brace were located within the Southern interior and exterior girders.

1.3 Numbering Nomenclature

All instrumentation was numbered and labelled in a Girder-Location-Instrument method. Girders were prefixed with the letter "G", and numbered G1 through G4, as shown in Figure A - 4, from North to South.

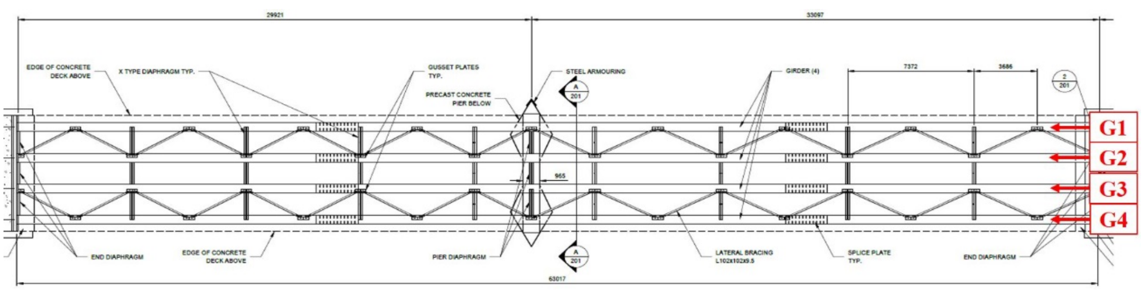

Figure A - 4. Girder Numbering Scheme.

Strain gauges used the prefix SG, and those on main girder locations were numbered SG1 through SG5 as shown in Figure A - 5. SG4 was always on the North side of the bottom flange. The location for the strain gauges are also dimensioned in Figure A - 6

Appendix A Instrumentation Details

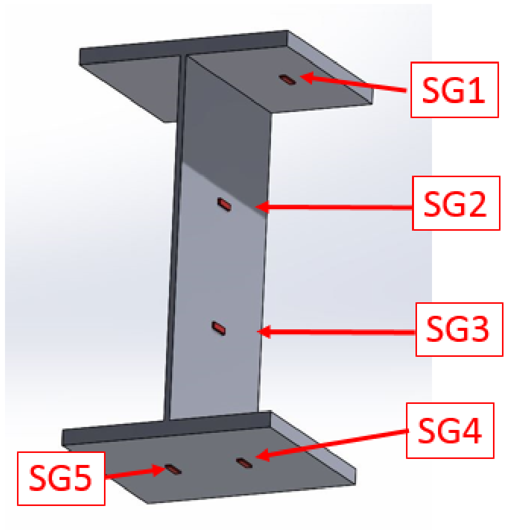

Figure A - 5. Main Girder Strain Gauge Locations and Nomenclature.

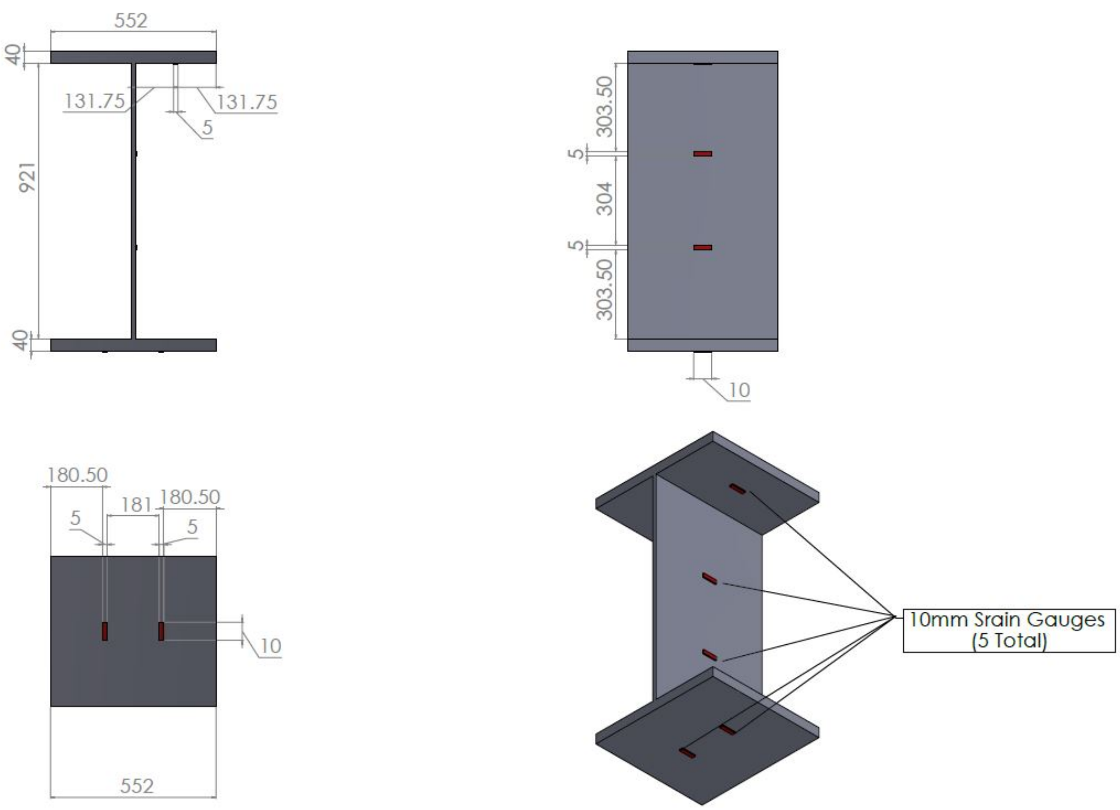

Figure A - 6. Main Girder Strain Gauge Locations (all dimensions mm).

Appendix A Instrumentation Details

Rosette Strain Gauges used the prefix RSG and were numbered RSG1 through RSG3 per girder (e.g. G1-RSG2) as shown in Figure A - 7. The RSG3 was always located 0.46 m (18 in) from the bottom flange. All Rosette Strain Gauges were located 1.3 m from the East bearings.

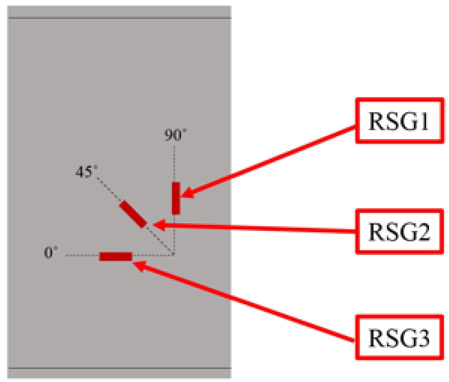

Figure A - 7. Rosette Strain Gauge Locations and Nomenclature.

Decking strain gauges used the prefix DSG and were numbered DSG1 through DSG8. They were installed on the underside of the corrugated steel decking between G2 and G3, and between G3 and G4, as shown in Figure A - 8.

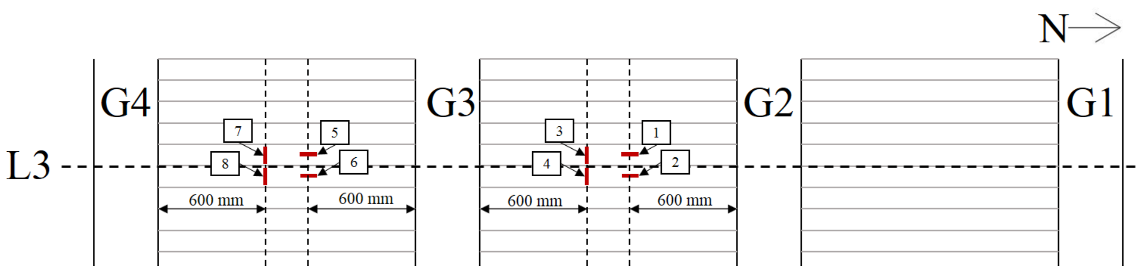

Figure A - 8. Decking Strain Gauge Locations and Nomenclature.

Appendix A Instrumentation Details

The X-Type diaphragm used the prefix XSG and its six gauges were numbered as shown in Figure A - 9.

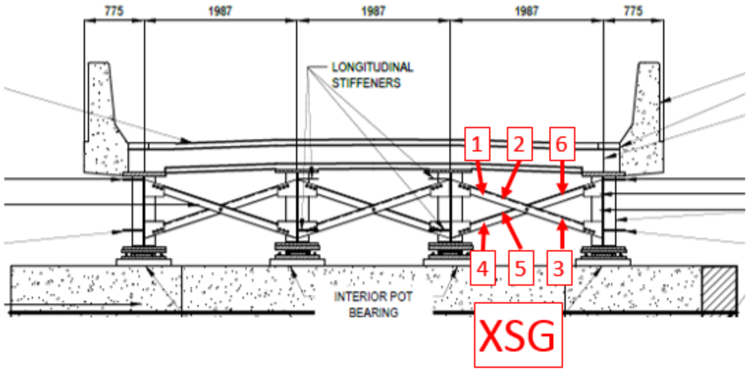

Figure A - 9. X-Type Diaphragm Strain Gauge Locations and Nomenclature.

The instrumented lateral brace used the prefix SSG and the three locations are shown in Figure A - 10.

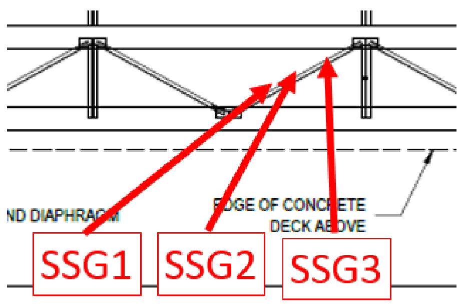

Figure A - 10. Lateral Brace Strain Gauge Locations and Nomenclature.

Appendix A Instrumentation Details

Wired linear position string potentiometers were installed to the underside of Girders, slightly offset from the strain gauges, at L1 and L3. G1 at L1 had no potentiometers installed. They were labelled SP2 through SP8 as shown in Figure A - 11. On the day of testing, several linear position traducers malfunctioned and as a result, there is only reliable data from SP3, SP5, and SP6.

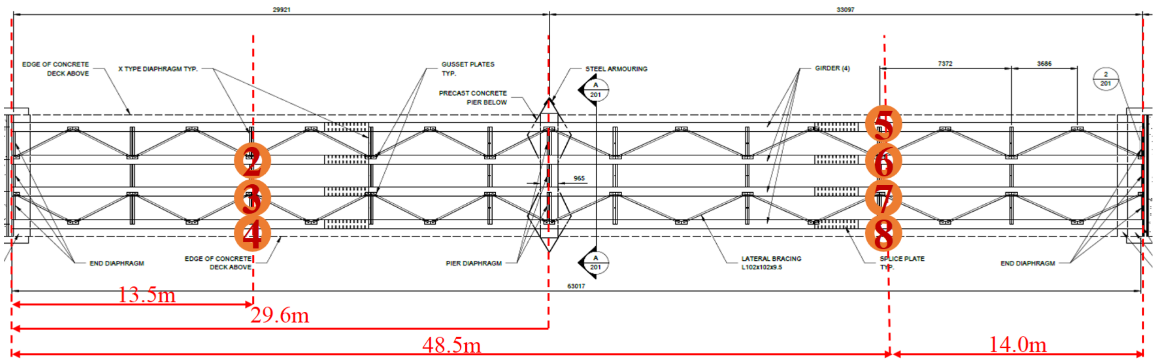

Figure A - 11. Wired Linear Position String Potentiometers.

The accelerometers were installed at the exterior edges of G1 and G4 at L1 and L3. The prefix used was "A", and reflected their Girder and Location position as per the standard nomenclature (e.g. G1-L1-A1).

A summary of all installed instrumentation, as per the nomenclature is provided in Table A - 1. Due to the number of channels required, three Data Acquisition (DAQ) Systems were used, and their locations split between L1 and L3.

Appendix A Instrumentation Details

Table A - 1. List of Instrumentation as per Nomenclature.

DAQ Location					
L1 DAQ IP: 192.168.0.14		L3 DAQ IP: 192.168.0.16		L3 DAQ IP: 192.168.0.13	
Channel	***Instrument***	***Channel***	***Instrument***	***Channel***	***Instrument***
1	G1-L1-SG1	1	G1-L3-SG1	1	DSG1
2	G1-L1-SG2	2	G1-L3-SG2	2	DSG2
3	G1-L1-SG3	3	G1-L3-SG3	3	DSG3
4	G1-L1-SG4	4	G1-L3-SG4	4	DSG4
5	G1-L1-SG5	5	G1-L3-SG5	5	DSG5
6	G2-L1-SG1	6	G2-L3-SG1	6	DSG6
7	G2-L1-SG2	7	G2-L3-SG2	7	DSG7
8	G2-L1-SG3	8	G2-L3-SG3	8	DSG8
9	G2-L1-SG4	9	G2-L3-SG4	9	G1-L3-SP5
10	G2-L1-SG5	10	G2-L3-SG5	10	G2-L3-SP6
11	G3-L1-SG1	11	G3-L3-SG1	11	G3-L3-SP7
12	G3-L1-SG2	12	G3-L3-SG2	12	G4-L3-SP8
13	G3-L1-SG3	13	G3-L3-SG3	13	G1-L1-A1
14	G3-L1-SG4	14	G3-L3-SG4	14	G4-L1-A2
15	G3-L1-SG5	15	G3-L3-SG5	15	G1-L3-A3
16	G4-L1-SG1	16	G4-L3-SG1	16	G4-L3-A4
17	G4-L1-SG2	17	G4-L3-SG2	17	
18	G4-L1-SG3	18	G4-L3-SG3	18	
19	G4-L1-SG4	19	G4-L3-SG4	19	
20	G4-L1-SG5	20	G4-L3-SG5	20	
21	G1-L2-SG1	21	G3-L2-SG1	21	
22	G1-L2-SG2	22	G3-L2-SG2	22	
23	G1-L2-SG3	23	G3-L2-SG3	23	
24	G1-L2-SG4	24	G3-L2-SG4	24	
25	G2-L2-SG1	25	G4-L2-SG1	25	
26	G2-L2-SG2	26	G4-L2-SG2	26	
27	G2-L2-SG3	27	G4-L2-SG3	27	
28	G2-L2-SG4	28	G4-L2-SG4	28	
29	G2-L2-SG5	29	G3-RSG1	29	
30	SSG1	30	G3-RSG2	30	
31	SSG2	31	G3-RSG3	31	
32	SSG3	32	G4-RSG1	32	
33	XSG1	33	G4-RSG2	33	
34	XSG2	34	G4-RSG3	34	
35	XSG3	35	G1-RSG1	35	
36	XSG4	36	G1-RSG2	36	
37	XSG5	37	G1-RSG3	37	
38	XSG6	38	G2-RSG1	38	
39	G3-L1-SP3	39	G2-RSG2	39	
40	G4-L1-SP4	40	G2-RSG3	40	

Appendix A Instrumentation Details

1.4 Site Photos

Various images from the installation and site setup are provided in Figure A - 12 through Figure A - 16.

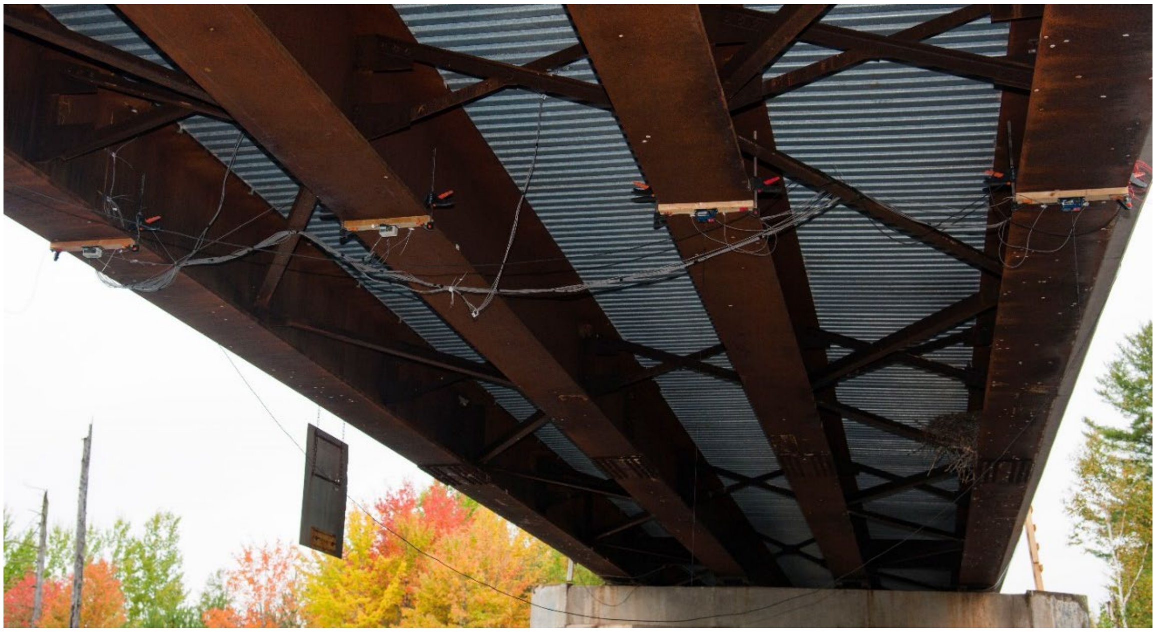

Figure A - 12. View of L3 Instrumentation from East Abutment.

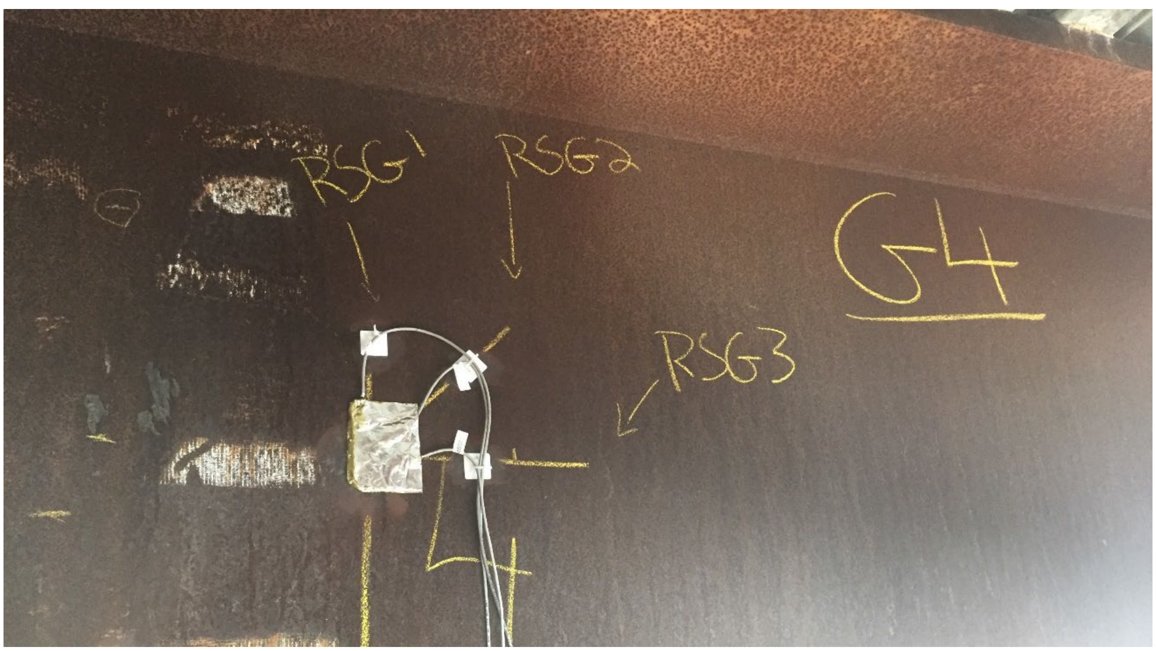

Figure A - 13. Installed Rosette on G4.

Figure A - 14. DAQ at L3 from Bridge Surface.

Figure A - 15. Computer Setup in Rear of Cube Truck.

Figure A - 16. View of Stochastic Pattern for DIC at L3.

Appendix B Platform Details

As the bridge spanned a waterway, it was necessary to develop a solution to install instrumentation near the mid-spans. Several options were explored including erecting of temporary scaffolding, installation from a boat, use of a bridge boom, or purchase of a hanging/rolling scaffold system. Purchase of the rolling scaffold system proved to be the most economical and reliable option. A scaffold system with components tailored to a steel girder bridge type was procured from Swing-Lo Suspended Scaffold Company. Photos of the scaffold system are shown in Figure B - 1 and Figure B - 2.

Figure B - 1. Platform Assembled in Lab.

Appendix B Platform Details

Figure B - 2. Platform in-use on Bridge.

Appendix C Smooth Surface Micro-Strain Values

The tables presented in Appendix C provide the maximum micro-strain results for all tested vehicles, at all three locations, for the smooth surface tests. The tests carried out are detailed in the left column, with the results per gauge indicated in the other columns. Use Appendix A to relate instrument numbering to bridge location.

1. Leopard 2

Table C - 1. Leopard 2 Smooth Surface Results at L1.

	Max ϵ (μm/m)							
	G1-L1-SG4	*G1-L1-SG5*	*G2-L1-SG4*	*G2-L1-SG5*	*G3-L1-SG4*	*G3-L1-SG5*	*G4-L1-SG4*	*G4-L1-SG5*
Centerline Crawl	115.71	114.19	138.93	115.95	132.21	121.92	98.37	101.11
Edge Crawl	102.76	95.48	134.55	108.46	136.03	125.50	124.24	126.71
Centerline - 10	94.80	89.25	124.71	105.83	135.58	124.72	122.18	126.60
Centerline - 20	111.93	116.02	131.78	112.41	129.48	119.92	96.27	99.34
Centerline - 30	108.03	111.97	135.15	114.36	145.53	134.13	105.41	113.31
Centerline - 40	107.36	105.36	131.42	108.82	137.53	122.02	97.04	100.75
Centerline - 50	91.17	101.11	125.34	104.23	140.70	127.13	104.54	106.78
Edge - 10	90.97	76.70	122.77	105.39	133.15	122.49	126.06	128.37
Edge - 20	83.53	77.41	121.12	103.08	139.51	124.88	128.20	133.48
Edge - 30	78.56	77.93	116.71	95.04	138.14	125.80	125.16	128.46
Edge - 40	80.93	88.06	132.76	102.70	146.79	134.68	129.99	132.68
Edge - 50	82.91	81.59	119.97	100.21	140.28	123.78	120.52	129.53

Table C - 2. Leopard 2 Smooth Surface Results at L2.

	Min ϵ (μm/m)				
	G1-L2-SG4	*G2-L2-SG4*	*G2-L2-SG5*	*G3-L2-SG4*	*G4-L2-SG4*
Centerline Crawl	-49.54	-55.73	-54.64	-67.13	-49.17
Edge Crawl	-47.91	-62.41	-50.58	-77.52	-66.28
Centerline - 10	-46.03	-63.67	-49.38	-66.68	-58.22
Centerline - 20	-58.98	-62.50	-57.34	-70.70	-54.12
Centerline - 30	-49.24	-61.91	-47.62	-67.51	-54.29
Centerline - 40	-52.24	-66.38	-59.01	-61.54	-52.93
Centerline - 50	-47.01	-64.49	-54.38	-65.49	-57.83
Edge - 10	-45.71	-56.83	-45.19	-79.48	-70.66
Edge - 20	-40.37	-61.45	-43.53	-70.60	-74.34
Edge - 30	-40.16	-58.27	-43.47	-70.88	-70.89
Edge - 40	-40.87	-57.85	-42.98	-84.34	-69.54
Edge - 50	-42.66	-59.44	-42.48	-88.49	-79.31

Table C - 3. Leopard 2 Smooth Surface Results at L3.

	Max ϵ (μm/m)							
	G1-L3-SG4	*G1-L3-SG5*	*G2-L3-SG4*	*G2-L3-SG5*	*G3-L3-SG4*	*G3-L3-SG5*	*G4-L3-SG4*	*G4-L3-SG5*
Centerline Crawl	116.44	115.62	136.87	136.51	125.67	130.81	103.68	109.90
Edge Crawl	88.44	82.70	125.68	126.02	137.78	143.28	139.63	145.83
Centerline - 10	103.59	93.91	133.67	131.19	134.68	137.99	132.36	140.63
Centerline - 20	118.53	118.42	140.78	141.48	132.05	139.93	97.75	113.57
Centerline - 30	105.98	102.23	135.58	134.61	133.07	138.41	118.87	126.34
Centerline - 40	116.53	111.41	137.58	135.66	130.65	135.56	103.90	114.20
Centerline - 50	99.71	93.82	132.87	131.82	136.41	139.48	123.45	135.64
Edge - 10	93.11	89.97	131.32	131.23	138.61	144.50	124.70	131.93
Edge - 20	88.03	78.81	127.30	124.81	141.70	146.48	143.38	150.08
Edge - 30	84.93	75.50	126.20	122.83	138.78	144.18	134.09	143.03
Edge - 40	88.13	85.48	125.88	124.62	138.75	140.77	146.75	149.82
Edge - 50	83.27	81.21	129.86	123.76	134.22	144.59	130.70	133.87

Appendix C Smooth Surface Micro-Strain Values

2. HLVW

Table C - 4. HLVW Smooth Surface Results at L1.

	Max ϵ (µm/m)							
	G1-L1-SG4	*G1-L1-SG5*	*G2-L1-SG4*	*G2-L1-SG5*	*G3-L1-SG4*	*G3-L1-SG5*	*G4-L1-SG4*	*G4-L1-SG5*
Centerline Crawl (W)	46.83	56.70	64.43	49.45	79.88	71.26	71.93	71.35
Centerline Crawl (E)	63.90	65.31	75.71	66.59	69.25	62.82	58.33	58.12
Edge Crawl	42.98	34.03	62.33	48.92	77.14	71.16	94.93	92.04
Centerline - 10	57.62	65.48	68.93	61.17	70.90	64.03	60.09	60.59
Centerline - 20	60.42	61.98	72.91	61.59	75.77	71.15	67.95	66.81
Centerline - 30	55.68	65.07	64.43	52.33	73.43	67.06	62.29	63.31
Centerline - 40	65.06	59.90	79.90	69.11	74.30	68.85	63.12	67.95
Centerline - 50	58.48	53.55	68.17	56.26	67.59	60.58	57.63	61.73
Edge - 10	40.48	41.26	59.28	45.83	84.07	72.82	91.37	91.28
Edge - 20	44.07	44.74	59.29	50.96	81.93	75.54	94.88	90.28
Edge - 30	37.64	35.28	59.39	48.71	74.78	66.73	86.09	86.18
Edge - 40	36.81	35.98	50.86	44.61	83.15	72.90	88.28	91.07
Edge - 50	42.18	42.94	58.29	48.85	74.95	67.64	81.88	83.56

Table C - 5. HLVW Smooth Surface Results at L2.

	Min ϵ (µm/m)				
	G1-L2-SG4	*G2-L2-SG4*	*G2-L2-SG5*	*G3-L2-SG4*	*G4-L2-SG4*
Centerline Crawl (W)	-29.80	-45.73	-34.83	-66.18	-50.48
Centerline Crawl (E)	-36.00	-46.53	-36.69	-59.67	-34.79
Edge Crawl	-18.98	-35.20	-24.46	-65.43	-65.55
Centerline - 10	-40.57	-50.98	-41.43	-58.17	-31.28
Centerline - 20	-34.16	-48.97	-35.94	-61.21	-49.83
Centerline - 30	-33.55	-47.19	-36.34	-58.76	-47.53
Centerline - 40	-40.02	-48.03	-38.88	-58.23	-46.75
Centerline - 50	-34.48	-47.41	-36.68	-67.77	-54.04
Edge - 10	-17.66	-38.18	-24.64	-59.22	-77.93
Edge - 20	-20.43	-35.94	-25.58	-69.14	-60.59
Edge - 30	-20.96	-37.20	-24.08	-59.35	-61.33
Edge - 40	-19.78	-41.44	-24.58	-58.86	-70.89
Edge - 50	-18.98	-35.00	-21.16	-57.62	-66.11

Table C - 6. HLVW Smooth Surface Results at L3.

	Max ϵ (µm/m)							
	G1-L3-SG4	*G1-L3-SG5*	*G2-L3-SG4*	*G2-L3-SG5*	*G3-L3-SG4*	*G3-L3-SG5*	*G4-L3-SG4*	*G4-L3-SG5*
Centerline Crawl (W)	9.62	55.68	71.68	70.65	76.08	74.58	76.66	78.12
Centerline Crawl (E)	10.41	66.08	83.27	85.58	82.09	79.71	72.61	80.71
Edge Crawl	10.14	36.03	60.82	57.31	86.71	79.63	102.49	102.68
Centerline - 10	11.33	79.77	87.07	87.93	78.67	74.56	65.90	73.43
Centerline - 20	9.90	61.54	79.38	83.46	83.13	80.07	75.98	80.69
Centerline - 30	9.28	60.52	74.76	77.20	77.60	78.97	74.58	81.11
Centerline - 40	8.92	67.58	84.09	80.11	80.62	80.28	73.27	77.11
Centerline - 50	11.09	63.38	84.47	87.18	77.93	76.28	72.68	72.98
Edge - 10	12.97	39.98	63.38	59.63	84.58	83.78	102.13	97.01
Edge - 20	10.11	41.11	61.83	58.38	89.75	79.73	102.49	102.63
Edge - 30	12.16	42.01	69.94	73.49	92.65	85.55	94.40	95.48
Edge - 40	11.83	41.53	66.00	68.98	91.30	87.58	102.94	104.04
Edge - 50	12.59	37.29	70.13	71.14	87.15	86.46	105.30	102.68

Appendix C Smooth Surface Micro-Strain Values

3. E-LAV

Table C - 7. E-LAV Smooth Surface Results at L1.

	Max ϵ (µm/m)							
	G1-L1-SG4	*G1-L1-SG5*	*G2-L1-SG4*	*G2-L1-SG5*	*G3-L1-SG4*	*G3-L1-SG5*	*G4-L1-SG4*	*G4-L1-SG5*
Centerline Crawl (W)	48.91	38.82	56.23	50.24	44.39	41.83	42.12	39.64
Centerline Crawl (E)	47.34	49.05	56.97	47.78	53.49	49.17	41.88	38.71
Edge Crawl	36.47	28.65	51.97	43.40	49.47	44.23	50.53	50.90
Centerline - 10	42.74	38.03	51.18	41.11	49.51	43.84	36.79	39.78
Centerline - 20	42.84	36.03	52.13	48.87	45.53	42.14	43.64	42.69
Centerline - 30	45.09	38.18	55.87	45.89	50.33	46.01	46.28	45.93
Centerline - 40	47.28	45.23	49.07	44.86	43.32	40.73	35.34	37.24
Centerline - 50	44.77	38.01	51.63	45.12	51.33	45.83	42.20	43.32
Edge - 10	33.68	36.44	48.49	38.63	53.15	48.32	46.00	45.83
Edge - 20	34.74	32.89	47.89	36.73	52.86	43.82	45.23	45.58
Edge - 30	22.81	30.91	35.77	29.90	53.93	48.50	53.49	52.21
Edge - 40	35.90	36.87	50.74	41.75	51.60	47.08	43.78	41.55
Edge - 50	30.43	32.56	45.26	36.45	54.89	46.38	49.91	49.65

Table C - 8. E-LAV Smooth Surface Results at L2.

	Min ϵ (µm/m)				
	G1-L2-SG4	*G2-L2-SG4*	*G2-L2-SG5*	*G3-L2-SG4*	*G4-L2-SG4*
Centerline Crawl (W)	-19.51	-23.51	-19.74	-30.72	-37.74
Centerline Crawl (E)	-19.26	-21.48	-18.55	-36.71	-31.25
Edge Crawl	-13.18	-20.71	-13.33	-31.71	-38.26
Centerline - 10	-20.82	-25.92	-22.28	-36.88	-25.88
Centerline - 20	-19.69	-24.66	-19.23	-34.13	-28.78
Centerline - 30	-17.40	-23.12	-18.55	-33.02	-29.38
Centerline - 40	-24.54	-28.60	-24.50	-29.16	-28.83
Centerline - 50	-19.83	-27.11	-20.90	-43.60	-35.00
Edge - 10	-15.35	-21.89	-16.72	-36.18	-34.45
Edge - 20	-16.80	-26.78	-21.26	-45.64	-40.43
Edge - 30	-15.26	-22.26	-16.47	-31.76	-40.10
Edge - 40	-16.93	-21.98	-17.60	-31.68	-38.26
Edge - 50	-15.55	-25.90	-21.08	-38.94	-34.98

Table C - 9. E-LAV Smooth Surface Results at L3.

	Max ϵ (µm/m)							
	G1-L3-SG4	*G1-L3-SG5*	*G2-L3-SG4*	*G2-L3-SG5*	*G3-L3-SG4*	*G3-L3-SG5*	*G4-L3-SG4*	*G4-L3-SG5*
Centerline Crawl (W)	7.29	36.50	54.39	51.76	47.28	47.76	42.03	39.84
Centerline Crawl (E)	9.58	38.44	52.73	52.90	54.73	50.89	42.17	49.58
Edge Crawl	5.64	25.75	49.17	58.47	57.38	52.03	58.45	60.58
Centerline - 10	8.38	42.66	55.73	62.99	50.37	47.62	37.33	40.76
Centerline - 20	9.03	37.07	55.20	61.33	51.34	49.71	41.99	44.93
Centerline - 30	7.46	35.33	52.38	58.75	56.43	52.39	46.53	49.66
Centerline - 40	4.48	45.68	54.72	57.62	44.48	46.13	42.15	39.12
Centerline - 50	8.87	38.82	58.98	61.60	52.98	50.43	42.08	42.96
Edge - 10	5.51	26.54	50.84	50.28	52.43	51.00	60.60	63.79
Edge - 20	8.57	31.97	59.13	64.71	54.35	50.73	48.91	48.67
Edge - 30	7.85	28.13	48.69	48.37	52.07	50.14	51.05	51.30
Edge - 40	6.14	32.33	55.60	51.63	48.43	47.28	48.09	43.56
Edge - 50	7.82	30.28	52.23	50.77	58.60	52.93	47.18	50.39

4. Cougar

Table C - 10. Cougar Smooth Surface Results at L1.

	Max ϵ (μm/m)							
	G1-L1-SG4	*G1-L1-SG5*	*G2-L1-SG4*	*G2-L1-SG5*	*G3-L1-SG4*	*G3-L1-SG5*	*G4-L1-SG4*	*G4-L1-SG5*
Centerline Crawl (W)	42.88	45.19	54.38	45.43	51.90	47.73	44.94	40.36
Centerline Crawl (E)	44.55	39.69	52.35	42.76	47.40	42.16	36.37	40.43
Edge Crawl	32.54	25.83	47.03	37.88	52.07	47.49	64.88	62.56
Centerline - 10	37.51	48.03	46.10	38.18	49.41	44.79	37.58	36.96
Centerline - 20	39.07	35.28	49.53	40.70	49.65	42.87	41.11	42.88
Centerline - 30	45.25	45.28	59.05	46.36	47.56	44.73	43.89	43.72
Centerline - 40	50.32	50.07	60.70	49.53	49.48	46.43	43.55	43.50
Centerline - 50	51.32	43.83	54.33	47.35	49.04	43.61	40.23	41.98
Edge - 10	34.34	29.55	48.00	36.18	54.84	50.68	54.77	55.58
Edge - 20	19.17	26.53	32.93	26.98	54.53	48.86	56.88	55.73
Edge - 30	27.27	25.77	40.10	34.04	51.05	47.12	55.53	57.18
Edge - 40	22.81	31.30	47.56	36.24	56.80	50.63	56.98	56.34
Edge - 50	29.22	32.41	47.82	36.61	57.75	51.70	63.04	60.53

Table C - 11. Cougar Smooth Surface Results at L2.

	Min ϵ (μm/m)				
	G1-L2-SG4	*G2-L2-SG4*	*G2-L2-SG5*	*G3-L2-SG4*	*G4-L2-SG4*
Centerline Crawl (W)	-20.16	-24.21	-19.92	-28.47	-25.71
Centerline Crawl (E)	-21.83	-26.72	-22.86	-33.86	-27.71
Edge Crawl	-11.77	-20.22	-14.16	-35.78	-38.25
Centerline - 10	-22.57	-25.75	-24.78	-34.88	-17.14
Centerline - 20	-19.04	-28.72	-22.45	-24.53	-25.99
Centerline - 30	-19.44	-23.68	-19.64	-32.28	-38.83
Centerline - 40	-20.28	-24.69	-22.98	-30.51	-26.66
Centerline - 50	-21.22	-27.81	-23.36	-36.21	-17.58
Edge - 10	-13.25	-22.31	-14.62	-31.04	-31.28
Edge - 20	-11.80	-21.00	-13.86	-37.25	-38.86
Edge - 30	-12.43	-21.28	-12.58	-39.28	-38.53
Edge - 40	-12.41	-20.00	-15.08	-42.75	-37.71
Edge - 50	-10.91	-18.59	-13.82	-30.91	-35.74

Table C - 12. Cougar Smooth Surface Results at L3.

	Max ϵ (μm/m)							
	G1-L3-SG4	*G1-L3-SG5*	*G2-L3-SG4*	*G2-L3-SG5*	*G3-L3-SG4*	*G3-L3-SG5*	*G4-L3-SG4*	*G4-L3-SG5*
Centerline Crawl (W)	6.69	38.80	52.58	58.99	48.26	51.85	48.32	51.27
Centerline Crawl (E)	7.19	41.65	50.73	51.99	46.38	45.78	41.27	45.59
Edge Crawl	6.31	19.49	43.03	37.09	58.65	53.54	66.67	64.82
Centerline - 10	7.80	46.00	56.79	56.11	52.43	48.08	44.33	49.77
Centerline - 20	5.38	38.78	47.88	46.68	51.76	49.83	47.18	52.71
Centerline - 30	6.28	37.73	53.36	50.23	46.38	48.12	43.48	40.41
Centerline - 40	7.83	40.79	53.15	58.23	48.25	49.36	49.99	51.54
Centerline - 50	6.98	43.75	55.00	54.48	50.28	47.40	44.93	50.17
Edge - 10	6.42	25.59	43.28	51.93	53.43	53.95	63.04	64.64
Edge - 20	6.04	22.18	43.79	47.20	55.19	52.57	59.99	59.94
Edge - 30	8.50	22.17	41.79	39.62	59.35	52.23	59.60	65.86
Edge - 40	9.34	25.93	46.97	47.38	58.69	55.45	68.85	70.69
Edge - 50	8.11	23.73	42.25	38.52	52.42	52.68	64.17	67.08

Appendix D Obstacle Surface Micro-Strain Values

The tables presented in Appendix D provide the maximum micro-strain results for all tested vehicles, at L3, for the obstacle surface tests. The tests carried out are detailed in the left column, with the results per gauge indicated in the other columns. Use Appendix A to relate instrument numbering to bridge location.

1. Leopard 2

Table D - 1. Leopard 2 Obstacle Surface Results at L3.

	Max ϵ (µm/m)							
	G1-L3-SG4	*G1-L3-SG5*	*G2-L3-SG4*	*G2-L3-SG5*	*G3-L3-SG4*	*G3-L3-SG5*	*G4-L3-SG4*	*G4-L3-SG5*
Centerline - 10 - Single High	14.11	114.23	146.92	141.73	143.76	145.72	125.00	128.48
Centerline - 20 - Single High	11.65	112.34	147.95	155.53	151.02	152.97	129.49	136.22
Centerline - 30 - Single High	16.00	123.68	165.58	163.95	146.16	151.67	109.39	120.00
Centerline - 40 - Single High	12.98	99.73	146.37	137.15	150.24	153.25	136.16	138.18
Centerline - 50 - Single High	14.04	121.60	174.35	165.27	154.45	162.92	113.28	122.86
Centerline - 10 - Double High	13.82	149.13	174.96	175.03	169.34	176.48	159.67	158.93
Centerline - 20 - Double High	16.33	127.13	170.80	167.76	159.53	163.30	140.37	153.13
Centerline - 30 - Double High	19.63	125.23	172.84	166.67	162.33	162.36	122.43	129.03
Centerline - 40 - Double High	16.77	135.05	195.14	177.69	171.13	172.97	138.38	144.01
Centerline - 50 - Double High	14.63	137.69	175.58	170.62	164.03	171.00	137.46	142.64
Edge - 10 - Double High	17.08	92.29	141.47	151.23	156.62	153.44	163.93	164.68
Edge - 20 - Double High	14.33	104.63	153.88	156.14	171.50	176.39	183.02	186.84
Edge - 30 - Double High	15.83	89.79	143.64	142.89	166.78	170.34	172.80	185.93
Edge - 40 - Double High	15.60	93.11	152.13	140.11	164.08	164.17	168.37	168.04
Edge - 50 - Double High	15.44	77.49	135.60	126.94	165.09	159.69	164.43	177.46
Centerline - 10 - Triple High	15.28	132.75	164.30	164.18	163.19	158.42	114.85	134.50
Centerline - 30 - Triple High	14.51	145.97	210.27	177.04	186.45	197.28	140.93	151.88
Centerline - 50 - Triple High	14.26	134.00	209.03	178.19	202.34	197.36	147.77	166.77

2. HLVW

Table D - 2. HLVW Obstacle Surface Results at L3.

	Max ϵ (µm/m)							
	G1-L3-SG4	*G1-L3-SG5*	*G2-L3-SG4*	*G2-L3-SG5*	*G3-L3-SG4*	*G3-L3-SG5*	*G4-L3-SG4*	*G4-L3-SG5*
Centerline - 10 - Double High	19.46	117.34	1`23.78	129.98	118.88	116.04	108.70	120.25
Centerline - 20 - Double High	23.01	182.94	175.83	171.78	170.47	168.23	171.76	182.65
Centerline - 30 - Double High	14.18	160.86	164.96	158.79	153.81	153.40	148.38	160.78

3. E-LAV

Table D - 3. E-LAV Obstacle Surface Results at L3.

	Max ϵ (µm/m)							
	G1-L3-SG4	*G1-L3-SG5*	*G2-L3-SG4*	*G2-L3-SG5*	*G3-L3-SG4*	*G3-L3-SG5*	*G4-L3-SG4*	*G4-L3-SG5*
Centerline - 10 - Double High	35.93	132.51	128.65	137.10	118.14	121.43	126.53	131.68
Centerline - 20 - Double High	15.06	87.85	109.31	111.41	101.57	101.01	86.88	95.07
Centerline - 30 - Double High	13.47	65.78	71.45	70.19	75.30	67.62	75.78	77.87

4. Cougar

Table D - 4. Cougar Obstacle Surface Results at L3.

	Max ϵ (µm/m)							
	G1-L3-SG4	*G1-L3-SG5*	*G2-L3-SG4*	*G2-L3-SG5*	*G3-L3-SG4*	*G3-L3-SG5*	*G4-L3-SG4*	*G4-L3-SG5*
Centerline - 10 - Double High	6.65	67.06	81.43	85.59	74.10	74.60	62.45	64.83
Centerline - 20 - Double High	8.31	57.72	74.35	78.42	64.30	68.41	64.35	67.36
Centerline - 30 - Double High	9.68	65.85	82.57	82.81	72.08	70.57	65.93	65.97

Appendix E Braking Test Micro-Strain Values

The tables presented in Appendix E provide the maximum micro-strain results for all tested vehicles, at L3, for the braking tests. The tests carried out are detailed in the left column, with the results per gauge indicated in the other columns. Use Appendix A to relate instrument numbering to bridge location.

1. Leopard 2

Table E - 1. Leopard Braking Test Results at L3.

	Max ϵ (μm/m)							
	G1-L3-SG4	*G1-L3-SG5*	*G2-L3-SG4*	*G2-L3-SG5*	*G3-L3-SG4*	*G3-L3-SG5*	*G4-L3-SG4*	*G4-L3-SG5*
Centerline - 10 - Brake	4.78	119.58	164.95	156.93	174.49	170.58	159.59	163.71
Centerline - 30 - Brake	14.01	129.33	156.56	156.89	156.60	155.69	145.54	151.23
Centerline - 50 - Brake	4.50	111.04	158.57	154.64	173.14	169.50	167.03	174.21

2. HLVW

Table E - 2. HLVW Braking Test Results at L3.

	Max ϵ (μm/m)							
	G1-L3-SG4	*G1-L3-SG5*	*G2-L3-SG4*	*G2-L3-SG5*	*G3-L3-SG4*	*G3-L3-SG5*	*G4-L3-SG4*	*G4-L3-SG5*
Centerline - 10 - Brake	11.61	65.28	90.47	96.89	84.65	83.08	70.08	71.66
Centerline - 30 - Brake	11.21	66.53	78.30	75.78	87.32	82.13	88.73	91.34

3. E-LAV

Table E - 3. E-LAV Braking Test Results at L3.

	Max ϵ (μm/m)							
	G1-L3-SG4	*G1-L3-SG5*	*G2-L3-SG4*	*G2-L3-SG5*	*G3-L3-SG4*	*G3-L3-SG5*	*G4-L3-SG4*	*G4-L3-SG5*
Centerline - 10 - Brake	7.13	49.24	57.57	58.22	52.74	48.97	43.53	50.11
Centerline - 30 - Brake	8.91	40.47	62.72	61.56	63.00	58.72	52.65	53.38

4. Cougar

Table E - 4. Cougar Braking Test Results at L3.

	Max ϵ (μm/m)							
	G1-L3-SG4	*G1-L3-SG5*	*G2-L3-SG4*	*G2-L3-SG5*	*G3-L3-SG4*	*G3-L3-SG5*	*G4-L3-SG4*	*G4-L3-SG5*
Centerline - 10 - Brake	8.33	39.03	51.76	58.03	55.67	52.11	45.19	56.02
Centerline - 30 - Brake	7.23	42.32	57.18	60.30	50.47	49.89	39.16	44.13

Appendix F Vehicle Specifications

This page contained restricted information that was redacted for the online publication.

Appendix F Vehicle Specifications

This page contained restricted information that was redacted for the online publication.

Appendix F Vehicle Specifications

This page contained restricted information that was redacted for the online publication.

Appendix F Vehicle Specifications

This page contained restricted information that was redacted for the online publication.

CURRICULUM VITA

Name:	Anthony Marc Josaphat Everitt
Place and date of birth	Vancouver, British Columbia 28 January 1991
Education	Royal Military College of Canada (2009-2013) Kingston, Ontario BEng in Civil Engineering, 2013
Experience	4 Wing Construction Engineering Squadron 41 Construction Engineering Flight Commander CFB Cold Lake April 2016 – July 2017
	Operation IMPACT JEngr – Plans Officer Ali Al Salem Air Base, Kuwait January 2017 – April 2017
	Wing Construction Engineering Engineering Officer CFB Cold Lake May 2015 – April 2016
	Wing Construction Engineering Operations Officer CFB Cold Lake May 2015 – April 2016
	Wing Construction Engineering Planning Officer CFB Cold Lake May 2015 – November 2016