

**A RECOMMENDED PROCEDURE FOR LOAD TESTING OF
CONCRETE SLAB BRIDGES IN NEW BRUNSWICK**

by

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ABSTRACT

In New Brunswick, concrete slab bridges often rate poorly in shear during an analytical load rating. However, these bridges typically do not show signs of distress in service. This report explores the use of load testing to refine the load rating capacity of these types of bridges. Drawing from the current state-of-the-art on load testing, a standardized procedure for performing diagnostic load tests on concrete slab bridges in New Brunswick was developed. In addition, a hypothetical diagnostic load test was carried out using finite element simulation of the St. Leonard C.N.R. Overhead (North) bridge in St. Leonard New Brunswick. The hypothetical analysis resulted in a 7% increase at the abutment and 87% increase at the pier in shear live load capacity compared to traditional analysis methods.

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TABLE OF CONTENTS

ABSTRACT.....	ii
ACKNOWLEDGEMENTS.....	iii
TABLE OF CONTENTS.....	iv
LIST OF TABLES.....	vi
LIST OF FIGURES	vii
1 INTRODUCTION	1
1.1 OVERVIEW	1
1.2 OBJECTIVES	1
1.3 REPORT STRUCTURE.....	1
2 LITERATURE REVIEW OF BRIDGE LOAD TESTING.....	3
2.1 INTRODUCTION	3
2.2 PROOF LOAD TESTS.....	3
2.3 DIAGNOSTIC LOAD TESTS	8
2.4 DATA COLLECTION	11
2.4.1 STRAIN	12
2.4.2 DISPLACEMENT	16
2.4.3 ROTATION	19
2.4.4 DYNAMIC	20
2.4.5 DATA ACQUISITION/VISUALIZATION.....	21
2.4.6 EMERGING TECHNOLOGIES	21
2.5 LOAD TESTING PER NORTH AMERICAN CODES	24
2.5.1 CANADA	24
2.5.2 UNITED STATES OF AMERICA	27
2.6 ADDITIONAL CONSIDERATIONS	30
3 PROPOSED LOAD TESTING PROCEDURE FOR CONCRETE SLAB BRIDGES IN NEW BRUNSWICK	32
3.1 INTRODUCTION	32
3.2 INITIAL INVESTIGATION	35

3.3 PRELIMINARY ANALYSIS	36
3.4 PLANNING	41
3.5 LOAD TESTING.....	42
3.6 REFINED ANALYSIS	45
3.7 SUMMARY	49
4 CASE STUDY – LOAD TESTING AND REFINED LOAD RATING OF THE ST. LEONARD C.N.R. OVERHEAD NORTH BRIDGE.....	50
4.1 INTRODUCTION	50
4.2 INITIAL LOAD RATING.....	53
4.3 HYPOTHETICAL LOAD TEST	57
4.3.1 LOAD AND LOAD PLACEMENT.....	57
4.3.2 PRELIMINARY ANALYSIS	59
4.3.3 SENSOR LAYOUT.....	65
4.3.4 SIMULATED LOAD TEST	66
4.4 REFINED LOAD RATING	68
4.5 CONCLUSION.....	79
5 SUMMARY	80
5.1 GENERAL CONCLUSIONS AND SUMMARY	80
REFERENCES	81
APPENDIX A: PROPOSED TESTING PROCEDURE.....	84
CURRICULUM VITAE	

LIST OF TABLES

Table 1 – Stop Criteria for Load Testing Concrete Bridges - Lantsoght et al. (2017b)	6
Table 2 – SFRAME Model Properties.....	54
Table 3 – Live Load Capacity Factor Results.....	57
Table 4 – Test truck placement for maximum load effect.....	59
Table 5 – Sensor Measurements	68
Table 6 – Comparison of deflections between load test and refined model	69
Table 7 – Adjusted Strains.....	73
Table 8 – Flexural Resistance Strains.....	74
Table 9 – Shear Resistance Strains	75
Table 10 – Dead Load Flexural Strains	75
Table 11 – Dead Load Shear Strains.....	75
Table 12 – Flexural Live Load Effects	76
Table 13 – Shear Live Load Effects	77
Table 14 – Refined Flexural Live Load Capacity Factors from Load Test.....	78
Table 15 - Refined Shear Live Load Capacity Factors from Load Test.....	78

LIST OF FIGURES

Figure 1 - Dapped end of girder – Zhou and Guzda (2020)	7
Figure 2 - Test trucks on I-95 - Zhou and Guzda (2020).....	7
Figure 3 - Load testing of bridge 098 – Sherman et al. (2020).....	10
Figure 4 - Load testing of Bridge 251 - Sherman et al. (2020).....	10
Figure 5 - Load testing of bridge in Florida – Dong et al. (2020)	11
Figure 6 – Example operation of foil strain gauge	13
Figure 7 - Typical foil strain gauge installation.....	14
Figure 8 – Typical strain transducers installation	15
Figure 9 – Typical vibrating wire strain gauge	16
Figure 10 – Typical displacement dial gauge set up.....	17
Figure 11 – Typical LVDT set up for displacement	18
Figure 12 – Typical total station set up to record movement in a bridge	18
Figure 13 – Typical tiltmeter installation.....	19
Figure 14 - Example LVDT sensor operation for dynamic measurements Etouney and Alampalli (2016).....	20
Figure 15 – Example DIC process – Lantsoght (2019b)	22
Figure 16 – Example FOS operation	23
Figure 17 – CAN/CSA S6 Load Test Flowchart	26
Figure 18 – AASHTO Diagnostic Load Test Flowchart	28
Figure 19 – AASHTO Proof Load Test Flowchart.....	29
Figure 20 – Load Testing Procedure Flow-Chart	34
Figure 21 – CL1, CL2, and CL3 Truck Configurations – CSA Group (2019a).....	38
Figure 22 – Typical test truck wheel layout.....	39
Figure 23 - Distribution of shear forces near a support – Lantsoght (2013).....	48
Figure 24 – Bridge Location	50
Figure 25 - Elevation view of St. Leonard C.N.R. Overhead (North).....	51
Figure 26 – Plan of St. Leonard C.N.R. Overhead (North)	51
Figure 27 – Typical Section of St. Leonard C.N.R. Overhead (North)	52
Figure 28 – Maximum Shear Force (CL1 loading, Uncracked)	55
Figure 29 – Maximum Shear Force (CL1 loading, Cracked)	55
Figure 30 – Maximum Moment (CL3 Loading, Uncracked)	56
Figure 31 – Maximum Moment (CL3 Loading, Cracked)	56
Figure 32 – Test truck axle configuration.....	58
Figure 33 – Typical NBDTI maintenance vehicle.....	58
Figure 34 – Plan of simplified FE model.....	60
Figure 35 – Isometric view of simplified FE model.....	60
Figure 36 – Max longitudinal strain on bottom of slab in Span 1 (Uncracked)	61

Figure 37 – Max longitudinal strain on bottom of slab in Span 2 (Uncracked)	61
Figure 38 – Max longitudinal strain on top of slab over Pier 1 (Uncracked)	62
Figure 39 – Max longitudinal strain on bottom of slab in Span 1 (Cracked)	62
Figure 40 – Max longitudinal strain on bottom of slab in Span 2 (Cracked)	63
Figure 41 – Max longitudinal strain on top of slab over Pier 1 (Cracked)	63
Figure 42 - Max transverse strain on bottom of slab in Span 1 (Cracked)	64
Figure 43 - Max transverse strain on bottom of slab in Span 2 (Cracked)	64
Figure 44 - Max transverse strain on top of slab over Pier 1 (Cracked)	64
Figure 45 – Sensor Layout	66
Figure 46 – 3D view of <i>The Bridge</i> model	67
Figure 47 – Plan view of <i>The Bridge</i> model	67
Figure 48 – YZ Shear Plane	70
Figure 49 – Shear strains at abutment	70
Figure 50 – Cross-section of shear strain taken at d_v from face of bearings at abutment.	71
Figure 51 – Shear strains at pier	71
Figure 52 - Cross-section of shear strain taken at d_v from face of pier	71
Figure 53 – Effective slab width per French load spreading method	72

1 INTRODUCTION

1.1 OVERVIEW

Over the life of a bridge, a wide range of load could be applied to the structure. In order to maintain a safe and efficient transportation network, owners often have a load rating performed on their bridges to ensure they are functioning as intended under ever-changing highway loads. In some cases, a load test is performed on a bridge to gain a better understanding of the behaviour.

1.2 OBJECTIVES

The New Brunswick Department of Transportation and Infrastructure has found that in some cases, bridges rate poorly through a traditional load rating but show no signs of distress. This is particularly true for concrete slab bridges. This objective of this report is to explore the use of load tests in refining load rating analysis of these concrete slab bridges. In addition, the report aims to present a proposed method of load testing these bridges to obtain a more accurate load rating.

1.3 REPORT STRUCTURE

The report consists of an introductory chapter, three main chapters, a conclusion chapters, as well as one appendix. Chapter 2 is a literature review of the current state-of-the-art on bridge load testing. It covers both Proof and Diagnostic load tests, common equipment used in load tests, as well as a review of how load testing is addressed in North American bridge codes. Chapter 3 takes the information from the literature review to develop a

proposed load testing procedure for concrete slab bridges. It covers initial investigation, planning and analysis; performing the load test; as well as using the test data to refine the load rating of the bridge. Chapter 4 takes the load testing procedure developed in Chapter 3 and applies it to a hypothetical load test. In lieu of performing a full-scale real-world load test, a highly detailed finite element model of an existing bridge was built and used instead. The results of the load test were used to calculate a more accurate load rating for the bridge. The proposed testing procedure is summarized and presented in the appendix.

2 LITERATURE REVIEW OF BRIDGE LOAD TESTING

2.1 INTRODUCTION

The practice of load testing bridges has been around as long as people have been building bridges. At a basic level, the original purpose of performing a load test was to prove that the bridge was able to safely carry the load. New bridges would have to pass a load test prior to opening to public use. This practice is still in use in Switzerland and Italy today (Lantsoght et al. 2017a).

Historically, the primary method of load testing a bridge was the proof load test. This involves applying known loads to a bridge to see if the bridge is able to support it without causing damage to the bridge. The goal of this method is to provide “proof” that the bridge can safely carry the loads that are expected to be applied to the bridge. In more recent years, an alternate method of load testing has been introduced, called a diagnostic load test. This method consists of subjecting a bridge to predetermined loads and recording the response in the bridge. The goal of this method is to verify the actual behaviour of the structure to allow for a more refined analysis of the structure. This could be in the form of validating the design assumptions for a new structure or determining the current behaviour of existing structures. This report will be focused on the use of diagnostic load testing on existing bridges.

2.2 PROOF LOAD TESTS

Proof load tests are a means to demonstrate that a bridge is able to support a given load. This type of test is well suited to validate that a bridge is safe before opening to general use. Another situation where a proof load test is useful is when the material properties

and construction of a bridge is unknown (Lantsoght et al. 2017). This is common for older concrete arch and slab type bridges where plans are no longer available leaving the concrete strength and internal reinforcing unknown.

Proof load tests are typically performed by applying a static load to a structure while sensors monitor for damages or non-linear responses up to a predetermined threshold. A general rule of thumb is for the proof load to be equal to twice the maximum allowable load (Lantsoght et al. 2017a). Alternatively, to correlate more closely with load and resistance factor designs, the proof load is based on the flexural moment required to match the flexural moment from the factored live load model (Lantsoght et al. 2017a).

AASHTO Manual for Bridge Evaluation recommends basing the proof load on the dead and live load (including dynamic load allowance) increased by a target live load factor.

The target live load factor, after modified for site conditions, is the value required to bring the load rating factor to 1.0 or greater (American Association of State Highway and Transportation Officials 2011). The test either stops once the target load has been reached or a stop-criteria has been met.

To avoid damages to the structure, the load is typically applied incrementally; whereas a load is increased, the test pauses until the structure response has stabilized, measurements and observations are taken, then the load can be further increased.

Lantsoght et al. (2017b) recommends that the load should be applied at a constant controlled rate between 3 kN/s – 10 kN/s and applied at four load levels:

- Level 1: a low load level (20-30% of Level 4) to check if all measurements function properly
- Level 2: the load level corresponding to the serviceability limit state
- Level 3: an intermediate level between Levels 2 and 4
- Level 4: the highest load level that has to be demonstrated by the proof load test, plus 5% to cover the uncertainties of proof load testing.

When performing a proof load test, it is critical to have accurate stop criteria defined prior to the start of the test. The stop criteria establish the boundary where unrecoverable damages could occur to the structure. As a result, the boundary should be set using conservative assumptions to avoid sudden failure modes. In general, proof load tests are stopped when non-linear behaviour is observed or visible signs of distress occur (American Association of State Highway and Transportation Officials 2011, Commander 2019). The Primer on Bridge Load Testing (Transportation Research Board 2019) notes that the stop criteria for steel bridges limits the total steel strain due to permanent loads and test loads to 90% of the yield strain. They also reference the German guidelines in *Deutscher Ausschuss für Stahlbeton* for stop criteria in reinforced concrete buildings:

- Limiting the sum of the concrete strain caused by the test load and the calculated strain caused by the permanent loads present at time of testing to 800 $\mu\epsilon$,
- Limiting the maximum concrete crack width to 0.5 mm (0.02 in.) for new cracks, and limiting the maximum increase in crack width to 0.3 mm (0.01 in.) for existing cracks,
- Limiting the residual crack width after a load cycle to 30% of the maximum crack width for a structure that was not cracked previously and to 20% of the maximum increase in crack width for a structure that was cracked previously.

Lantsoght et al. (2017b) proposed a set of stop criteria for concrete bridges in flexure and shear based on the concrete strain, crack width and changes in stiffness and is shown in Table 1.

Failure mechanism	Previously cracked in bending moment or not?	
	Uncracked	Cracked
Flexural failure	Concrete strains (Eq. (4))	Concrete strains (Eq. (4))
	$w_{max} \leq 0.5 \text{ mm}$	$w_{max} \leq 0.5 \text{ mm}$
	$w_{res} \leq 0.1 \text{ mm}$	$w_{res} \leq 0.1 \text{ mm}$
	$w_{res} < 0.3w_{max}$	$w_{res} < 0.2w_{max}$
	Stiffness reduction $\leq 25\%$	Stiffness reduction $\leq 5\%$
	Deflection profiles	Deflection profiles
Shear failure	Load-deflection graph	Load-deflection graph
	Concrete strains (Eq. (4))	Concrete strains (Eq. (4))
	Stiffness reduction $\leq 5\%$	Stiffness reduction $\leq 5\%$
	Deflection profiles	Deflection profiles
	Load-deflection graph	Load-deflection graph

Table 1 – Stop Criteria for Load Testing Concrete Bridges - Lantsoght et al. (2017b)

Zhou and Guzda (2020) carried out a proof load test on a prestressed concrete girder bridge on I-95 in Rhode Island, USA. The test specifically investigated shear at dapped ends of the girders where traditional analysis indicated insufficient capacity (Figure 1). The bridge was loaded with two 100,000 lb trucks travelling side-by-side (Figure 2). The test found the shear in the dapped ends under the test load to be greater than that resulting from the legal rating vehicles in AASHTO without any signs of non-linearity or distress.



Figure 1 - Dapped end of girder – Zhou and Guzda (2020)



Figure 2 - Test trucks on I-95 - Zhou and Guzda (2020)

Saraf and Nowak (1998) carried out proof load tests on three simple-span steel girder bridges in Michigan. These tests used two 55-ton M-60 military tanks for the test load. The applied moments from the tanks were more than 3.5 times greater than that generated by the AASHTO HS20 and 2.3 times greater than the two-unit 11-axle vehicle which was the highest legal load allowed in Michigan at the time. The tests found that the bridges reached the proof load with minimal distress or non-linearity indicating that the bridges are able to safely support the highest possible legal highway loads.

2.3 DIAGNOSTIC LOAD TESTS

A diagnostic load test provides a means to verify the behaviour of a structure such as stiffness, lateral load distribution, composite action, deflection, and rotations (Lantsoght 2019a). “The goal of a diagnostic test is not to test the load capacity directly, but to measure structural deformations for a set of applied load conditions” (Commander 2019). The diagnostic load test is well suited for bridges that could be subject to a variety of loads. In this case, the structure can be analyzed for any number of live load configurations through a refined analysis based on the actual bridge behaviour rather than performing a new proof load test for each new load configuration. The use of a diagnostic load test will frequently result in an improvement in the evaluation of the structure. This is due to the removal of conservative assumptions found in simplified analyses as well as the inclusion of beneficial factors such as secondary stiffening effects, unintended composite action, and actual load distribution (Sherman et al. 2020). Diagnostic load testing becomes particularly useful in structures that have increased

complexity and multiple load paths. The more complex the structure, the more likely a simplified analysis will produce incorrect results (Commander 2019).

In general, a diagnostic load test consists of instrumenting the structure, applying a known test load, and recording the structure's response. The test can be performed with static loading or dynamic loading depending on the type of information required from the test. Dynamic load testing goes beyond static loading to help establish the dynamic load allowance and natural frequencies of the structure. After the test is complete, the measurements are used to refine an analysis of the structure (American Association of State Highway and Transportation Officials 2011, Commander 2019, Lantsoght 2019a, Transportation Research Board 2019).

The test load used during a diagnostic load test is typically much lower than the capacity of the bridge but large enough to produce a measurable response. The bridge is most commonly loaded with loading vehicles or a system of hydraulic jacks. A specialized bridge load testing vehicle was developed in Germany called BELFA that allows for testing of spans up to 20 m (Lantsoght 2019a). Research is underway the Queen's University in Ontario into developing a rolling load simulator for testing bridges in Canada (Fam and Brennan 2019). However, loading vehicles are generally preferred as they allow for quick loading, unloading, and load positioning. Loading vehicles also allow for performing both static and dynamic load tests (Lantsoght 2019a).

Sherman et al. (2020) carried out diagnostic load tests on two bridges in Indiana: a 20 m single-span steel girder bridge with a reinforced concrete deck (Bridge 098, Figure 3) and a 24 m single-span steel pony truss with a reinforced concrete deck (Bridge 251, Figure 4). After the load tests were carried out on both bridges, a revised load rating was also

performed and compared with previous load rating results. The rating factor for Bridge 098 increased by 273%, while Bridge 251 experienced a 170% increase in load rating. The increase in capacity was attributed to the actual load distribution and unintended composite action between the deck and superstructure where the deck was not designed to be composite with the girders, as well as additional stiffness of secondary components such as the barrier system.



Figure 3 - Load testing of bridge 098 – Sherman et al. (2020)



Figure 4 - Load testing of Bridge 251 - Sherman et al. (2020)

In addition, Dong et al. (2020) carried out a diagnostic load test on a multi-span prestressed concrete girder bridge with reinforced concrete deck in Florida (Figure 5). After the load test was completed, the actual distribution factors and load rating factors were calculated and compared with those from those calculated through AASHTO standards. Both results were found to be more conservative than the AASHTO results.

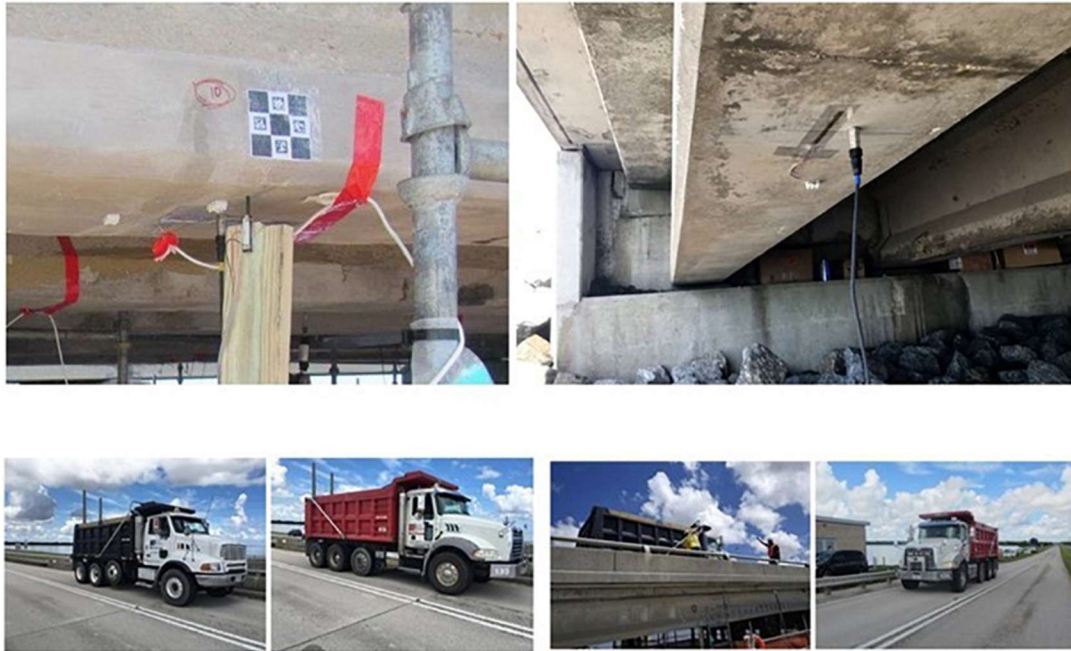


Figure 5 - Load testing of bridge in Florida – Dong et al. (2020)

2.4 DATA COLLECTION

During a load test, data will be collected on several different areas of the bridge. The data will be used to monitor the structure during the test and also, in the case of diagnostic load testing, be used to further refine an analysis of the structure afterwards. Ideally, every possible area of the structure would be instrumented to fully observe the structure's response as the load moves through the components. Evidently this is not a practical approach and measurements are usually limited to only those necessary to accurately

model the structure's behaviour. The types of measurements taken will depend on the type of structure being tested. In general, the types of measurements taken consist of stress or strain, displacement, rotation, and dynamic response (Transportation Research Board 2019). All of these parameters can be measured with a wide array of sensor types; however, only a select number of these can be post-installed to measure existing bridges. In addition to sensors to read the response, data acquisition (DAQ) and visualization equipment are required to receive and record the data.

2.4.1 STRAIN

Strain measurements are typically taken on main load carrying members to observe their response to the applied load. Common instruments used to measure strains are the foil strain gauge, demountable strain transducers, and vibrating wire strain gauges (American Association of State Highway and Transportation Officials 2011).

Foil strain gauges are a very common type of fully bonded to the structure either through welding or adhesives. Foil strain gauges work by applying a current through the gauge and when the structure that they're attached to moves, the change in electrical resistance in the circuit is interpreted as a change in strain in the structure, as seen in Figure 6.

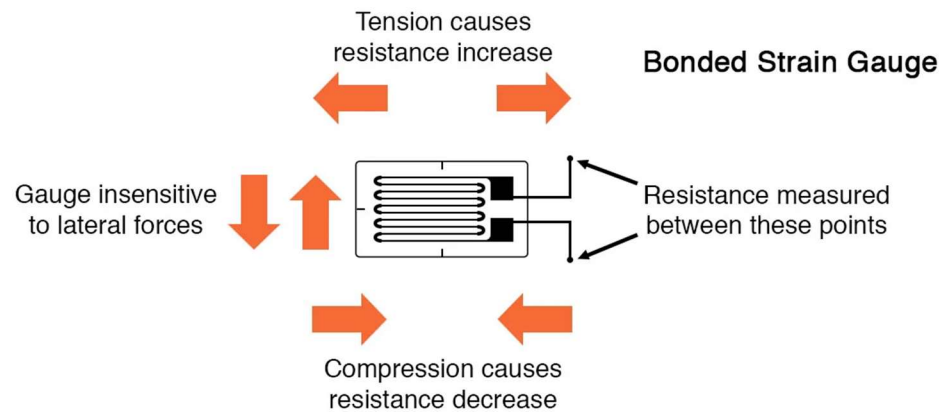


Figure 6 – Example operation of foil strain gauge¹

The most common type of circuit used in strain gauges is the Wheatstone bridge. Foil gauges are fairly easy to install given a suitable surface is available, they come in a wide variety of sizes and shapes and are generally inexpensive. However, they are single-use where they bond permanently with the structure, the installation surface may require extensive preparation, and they require a power source to provide outputs (Ettouney and Alampalli, 2016). An example of foil strain gauge installed on a truss member can be seen in Figure 7 below.

¹ <https://www.allaboutcircuits.com/textbook/direct-current/chpt-9/strain-gauges>



Figure 7 - Typical foil strain gauge installation²

Demountable strain transducers consist of a fixed enclosure containing a series of bonded strain gauges. These have two mounting points, one at each end of the sensor, with a known distance between the two that attaches to the structure. Changes in the distance between the two mount points are related to the strain in the structure. These sensors are reusable, installation is simple, and the cost can be spread over several uses.

Unfortunately, due to their size they may not fit in tight spaces where strains may be highest and they require accurate re-calibrations for each use (Ettouney and Alampalli 2016). A typical demountable strain transducer is shown in Figure 8 below.

² <https://www.resensys.com/r20/wireless-sensor-document-application/structural-fatigue-analysis-based-on-rainflow-counting-methodology.html>



Figure 8 – Typical strain transducers installation³

Vibrating wire strain gauges are a variation on the above strain gauge concept. Two anchor blocks are attached to the structure with a wire between them. At rest, the wire has a known natural frequency; however, when the wire is put in tension, the frequency of the wire changes. Instead of measuring the change in distance or resistivity, the change in frequency is related to the strain in the structure. These have similar advantages and disadvantages to the demountable strain gauges (Ettouney and Alampalli 2016). A vibrating wire strain gauge schematic can be viewed in Figure 9 below.

³ <https://bditest.com/product/st350-strain-transducer>



Figure 9 – Typical vibrating wire strain gauge⁴

The Primer on Bridge Load Testing advises that sensors with long gauge lengths should be used on reinforced concrete bridges. Smaller gauge sensors can be affected by the heterogeneous nature of concrete and local cracking, resulting in strain variations (Transportation Research Board 2019).

2.4.2 DISPLACEMENT

Displacement measurements are often taken at the mid span of the structure to record the deflected shape of the bridge under load. Displacements are most often measured with either dial gauges (Figure 10) or linear variable differential transformers (LVDT) (Figure 11) (American Association of State Highway and Transportation Officials 2011). Both sensors operate in a similar way where one end is fixed while the other moves with the movement of the structure. For a LVDT, the movement is measured through changes in

⁴ <https://rstinstruments.com/vibrating-wire-strain-gauge>

the voltage in the unit caused by the position of the inner rod in relation to internal coils. For a dial gauge, the movement is measured by the rotation of a series of springs and gears (Nijjaawan and Nijjaawan 2021). The downside to dial gauges is that they require someone to be present to read the gauge, where LVDT sensors can provide readings remotely. LVDT sensors have been proven to be reliable and durable. The downside to these sensors is that they require a fixed end to anchor the sensor to which may be difficult depending on what the bridge is spanning (Ettouney and Alampalli 2016).



Figure 10 – Typical displacement dial gauge set up⁵

⁵ <https://resolutionengineering.co.th/en/post-tension-slab-load-testing>



Figure 11 – Typical LVDT set up for displacement⁶

An alternative option to these sensors is to measure movement through the use of a total station. This requires the installation of a reflective target at measurement points. Similar to dial gauges, the main drawback to this method is the time it takes to individually collect each targets movement. While the readings can all be taken from the same position, unlike for dial gauges, it also requires a direct line of sight to each sensor (Lantsoght 2019a). A typical total station set up is shown in Figure 12.

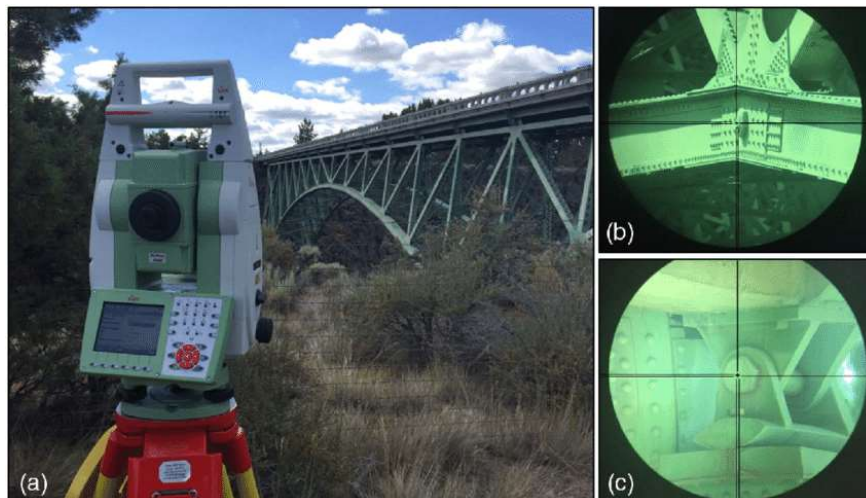


Figure 12 – Typical total station set up to record movement in a bridge⁷

⁶ <https://bditest.com/product/lvdt>

2.4.3 ROTATION

Rotation measurements are typically taken near supports through the use of tiltmeters or inclinometers. These types of sensors measure rotation by a vial of fluid, similar to a carpenter's level, with electrodes at each end and when more fluid touches one electrode the difference is measured with an electric output (Figure 13).



Figure 13 – Typical tiltmeter installation⁸

It is worth noting that they measure the rotation of the sensor relative to gravity and not necessarily the unloaded position of the structure. These can be difficult to install and due

7

https://www.researchgate.net/publication/319373638_BridgeDex_Proposed_Web_GIS_Platform_for_Managing_and_Interrogating_Multiyear_and_Multiscale_Bridge-Inspection_Images

⁸ <https://www.sisgeo.com/products/inclinometers-and-tiltmeters/item/d-tiltmeter-digital-mems-tiltmeters.html>

to their relatively slow reaction time, they are not well suited for dynamic measurements. On the other hand, they do not require additional interpolation of the data and are able to measure very small angles (Ettouney and Alampalli 2016, Jewell Instruments 2021).

2.4.4 DYNAMIC

Dynamic measurements are collected during a dynamic load test with LVDT accelerometers. These function similarly to the displacement LVDT sensors except the inner rod is attached to a moving mass and spring of known mass and stiffness respectively (Figure 14).

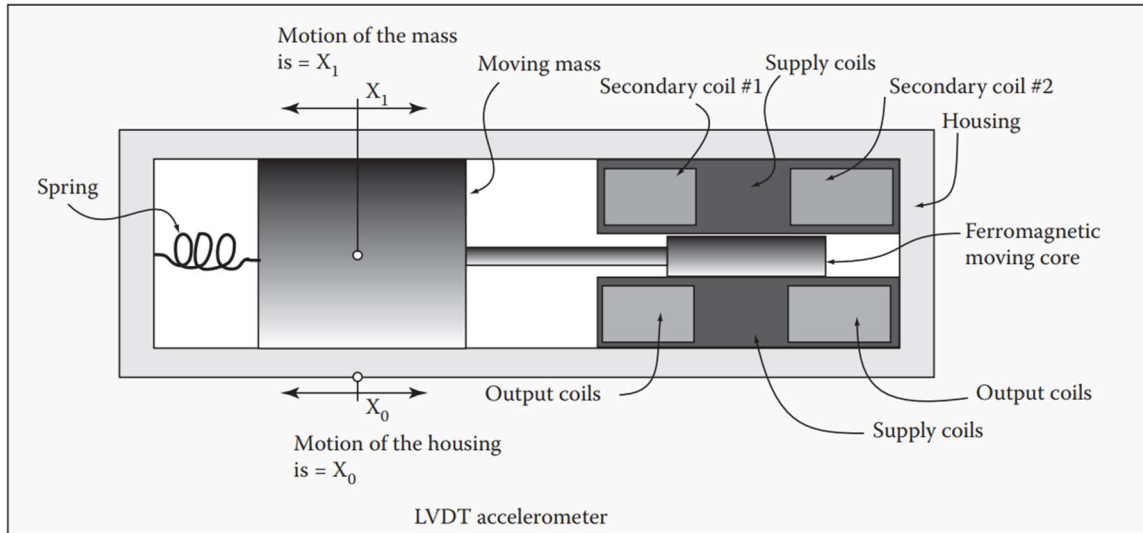


Figure 14 - Example LVDT sensor operation for dynamic measurements

Ettouney and Alampalli (2016)

The standard equation of dynamic motion (with the mass, damping, and stiffness parameters known) is used with the movement of the spring proportional to the voltage output of the sensor. These carry all the same advantages and disadvantages as the

displacement LVDT sensors (American Association of State Highway and Transportation Officials 2011, Ettouney and Alampalli 2016).

2.4.5 DATA ACQUISITION/VISUALIZATION

In addition to all of the measurements taken, systems of data acquisition and visualization are also needed. The DAQ selected for the test will depend on the data being collected and the required sampling rate to capture the desired response of the structure. Where active monitoring of the test data is required to ensure test safety, the DAQ and visualization system must be able to deliver real-time output (Lantsoght 2019a).

2.4.6 EMERGING TECHNOLOGIES

Digital Image Correlation (DIC) is an emerging technology in the remote sensing field. High resolution cameras take a series of images of the element being tested while specialized, often proprietary, software tracks the movement of pixels in the images and interprets them as strains or deformations depending on the goal of the test. The test usually involves the application of a special speckled spray to the surface of the member, allowing the software to track the movement of the speckles. An example test setup is shown in Figure 15 below.

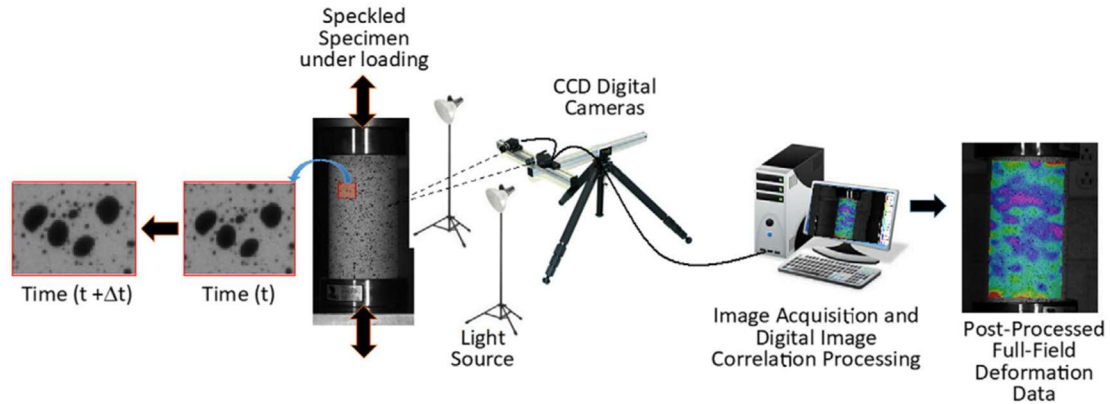


Figure 15 – Example DIC process – Lantsoght (2019b)

At the moment, DIC remains expensive compared to traditional sensors but as improvements in camera resolutions, software capabilities, and computer processing power increase, DIC will likely become a more commonly used tool. The main advantage of DIC is that it is able to detect changes in any direction over a wider area compared to discrete sampling points with single sensors. On the downside, this method does not allow for real-time monitoring of the structure. DIC would need to be paired with additional sensors during a load test (Commander 2019, Lantsoght 2019b).

Another upcoming technology is fibre optic sensors (FOS). FOS consist of an internal core by which a signal pass through. External changes to the sensor affect the passage of the signal such as changes to the wavelength or scattering of the signal. There are three main types of sensors: grating-based sensors, interferometric sensors, and distributed sensors; each with their own method to interpret the changes to the signal. An illustrated example of a FOS is shown in Figure 16.

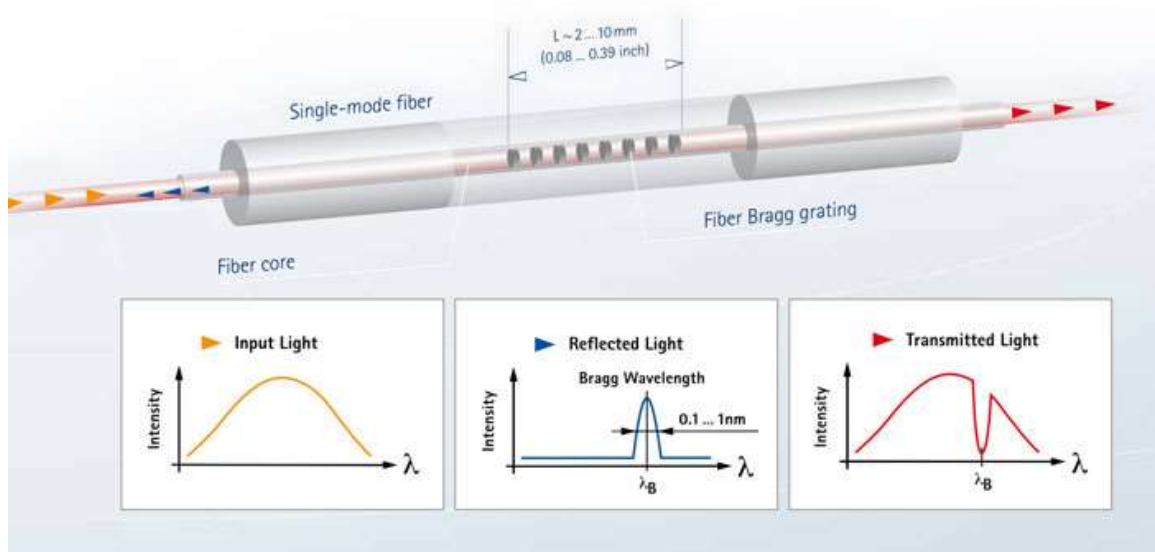


Figure 16 – Example FOS operation⁹

FOS are most commonly installed during construction for long term health monitoring of the structure; however, they can also be attached to the exterior of the structure for observing the response of existing structures. These sensors can be installed over a relatively short distance similar to traditional sensors, but the great benefit of FOS is that they can be installed over the entire length of the structure effectively acting as a series of discrete sensors in a row. These types of sensors are known as distributed fibre optic sensors (DFOS) and are quickly becoming the popular choice for load tests. DFOS are able to monitor strain, temperature, and deflection information along the length of the sensor. Of particular interest, is the ability to measure deflections without the need for a

⁹ <https://www.hbm.com/en/6827/article-how-does-an-optical-strain-gauge-actually-work>

ground reference. Some of the benefits of FOS are resistant to external electrical influences, ability to fit into small spaces, and durability in aggressive environments. The cost of using FOS has been relatively high compared to traditional sensors. With an increased use of fibre optics in other fields, the cost of using FOS is decreasing and becoming comparable with other sensor options (Del Grosso et al. 2001, Lantsoght 2019b).

2.5 LOAD TESTING PER NORTH AMERICAN CODES

After a load test has been completed, the measured response can be used to refine the analysis of the structure. The Canadian Highway Bridge Design Code (S6) and AASHTO Manual for Bridge Evaluation are the main two methods for load rating bridges in North America. Mexico does not currently have their own evaluation guideline but frequently adopt the AASHTO guidelines for load testing. Each publication provides their own approach to load testing of bridges and how the results are used to load rate a bridge.

2.5.1 CANADA

Section 14.16 of S6 (CSA Group 2019a) provides the requirements for load testing and rating of bridges in Canada. Bridges can only be load tested after a theoretical evaluation has been completed according to Section 14. The bridge must be instrumented and monitored during the test for any potential failure or damages as well as the behaviour of the structure. For a static load test, the load must be incrementally increased until the target load is reached, damages begin to occur, or if non-linear behaviour is observed. Dynamic testing of the structure is also permitted. The testing loads used during the load test may simulate effects of the CL evaluation vehicle or a specified Permit Controlled

vehicle but should be applied so as to subject the critical components to maximum load effects.

The commentary for S6 (CSA Group 2019b) provides a method to use the load testing results to determine the live load capacity factor (LLCF) for the bridge being evaluated.

During the load test, two situations can occur, leading to two different means to calculate the LLCF for the bridge. Situation 1 is when the load test is stopped when the testing load has been reached without causing damage to the structure. In this situation, the results of the test are extrapolated up to the S6 evaluation load. Extrapolation is not recommended when the primary members are susceptible to buckling as the observed behaviour under the lower testing load may not accurately represent the behaviour at higher loads. This is done through Eq. 1 below.

$$F = \frac{(\epsilon_e - \epsilon_{DL}) \frac{L_{test\ truck\ theor}}{\epsilon_{test\ truck\ measure}}}{L_{CHBD\ truck\ theor} \alpha_L (1 + l)} \quad \text{Eq. 1}$$

Where:

F	Live load capacity factor
ϵ_e	Factored strain due to member resistance
ϵ_{DL}	Factored strain due to permanent loads
$L_{test\ truck\ theor}$	Theoretical load effect due to test truck
$\epsilon_{test\ truck\ measure}$	Measured strain due to test truck from load test
$L_{CHBD\ truck\ theor}$	Theoretical load effect due to CHBD evaluation truck
α_L	Live load factor
l	Dynamic load allowance

Situation 2 is when the load test is stopped before the testing load has been reached due to the observation of damages or non-linear behaviour in the structure. In this case, the load that was reached before ending the test can be considered to be the load carrying

capacity, inadvertently becoming a low-level proof load test. The LLCF can then be calculated through Eq. 2.

$$F = \frac{L_{test\ truck\ theor}}{L_{CHBD\ truck\ theor} \alpha_L (1 + l)} \quad \text{Eq. 2}$$

Where:

F	Live load capacity factor
$L_{test\ truck\ theor}$	Theoretical load effect due to test truck
$L_{CHBD\ truck\ theor}$	Theoretical load effect due to CHBD evaluation truck
α_L	Live load factor
l	Dynamic load allowance

This equation effectively ratios the theoretical results from the test to the theoretical results from S6 analysis. As with Situation 1, the theoretical calculations can include observed behaviour such as observed lateral distribution. The load testing procedure is demonstrated in Figure 17 below.

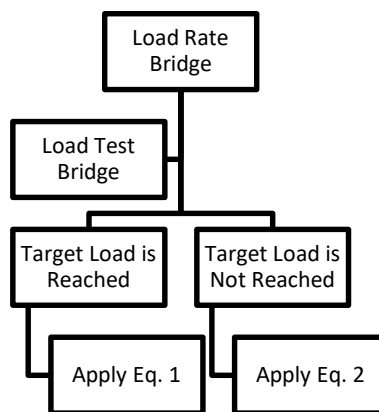


Figure 17 – CAN/CSA S6 Load Test Flowchart

2.5.2 UNITED STATES OF AMERICA

AASHTO Manual for Bridge Evaluation Section 8 (American Association of State Highway and Transportation Officials 2011) offers the methods for evaluation of a bridge through load testing. Unlike the Canadian code, the American manual provides a means to rate a bridge with a proof load test and diagnostic load test.

For a diagnostic load tests, Section 8.8.2 directs the bridge to be first load rated in accordance with A6.4.2.1 using Eq. 3 below. Once a standard load rating has been completed, the load test results are used to establish the adjustment factor K . This adjustment factor comprises of two parts, K_a and K_b . According to the Manual for Bridge Evaluation (American Association of State Highway and Transportation Officials 2011) “ K_a accounts for the benefit derived from the load test, if any, and consideration of the section factor (area, section modulus, etc.) resisting the applied load.” K_a also removes the benefits from composite action for a non-composite member. As stated by Lantsoght (2019a) “ K_b takes into account differences between the actual behaviour of the bridge and the revised analytical model, specifically with regard to lateral and longitudinal load distribution and the participation of other members”. For the refined load rating value, the standard load rating result is modified with the adjustment factor K as shown in Eq. 4. A rating factor greater than 1.0 indicates that the bridge is able to support the live load. The diagnostic load testing procedure is demonstrated in Figure 18 below.

$$RF_c = \frac{C - (\gamma_{DC})(DC) - (\gamma_{DW})(DW) \pm (\gamma_P)(P)}{(\gamma_{LL})(LL + IM)} \quad \text{Eq. 3}$$

$$RF_T = RF_c \times K \quad \text{Eq. 4}$$

Where:

RF_c	Load rating factor based on calculations prior to incorporating test results
C	Capacity
γ_{DC}	LRFD load factor for structural components and attachments
DC	Dead load effect due to structural components and attachments
γ_{DW}	LRFD load factor for wearing surfaces and utilities
DW	Dead load effect due to wearing surfaces and utilities
γ_P	LRFD load factor for permanent loads other than dead loads = 1.0
P	Permanent loads other than dead loads
γ_{LL}	Evaluation live load factor
LL	Live load effect
IM	Dynamic load allowance
K	Adjustment factor
RF_c	Load rating factor for live load capacity based on load test results

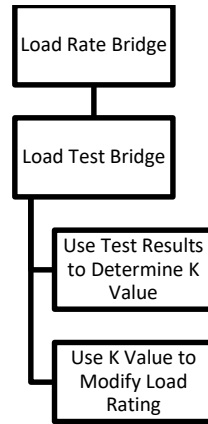


Figure 18 – AASHTO Diagnostic Load Test Flowchart

Rather than calculating the capacity of a bridge to carry live load in addition to dead load, the proof load results prove the ability of the bridge to carry the dead load and a magnified live load. In this situation, Eq. 3 above is worked backwards to determine the required live load to produce a rating equal or greater than 1.0. This live load becomes

the target proof load to be applied to the bridge during testing. The target proof load is calculated with Eq. 5 where the required live load is modified with the coefficient X_{pA} from Eq. 6. X_{pA} begins at 1.4 and is modified to range from 1.3 and 2.2 depending on: if one-lane loading controls the response, if the structure has fracture-critical details, the condition of the structure, and if the structure is ratable. The proof load testing procedure is demonstrated in Figure 19 below.

$$L_T = X_{pA} L_R (1 + IM) \quad \text{Eq. 5}$$

$$X_{pA} = X_p \left(1 + \frac{\Sigma \%}{100}\right) \quad \text{Eq. 6}$$

Where:

L_T	Target proof load
X_{pA}	Target adjusted live load factor
L_R	Unfactored live load of the rating vehicle
IM	Dynamic load allowance
X_p	Target live load factor

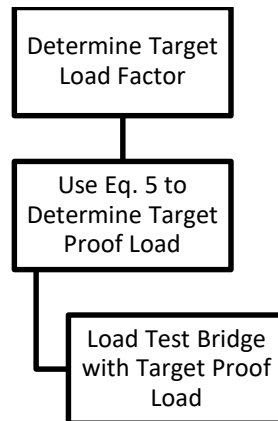


Figure 19 – AASHTO Proof Load Test Flowchart

2.6 ADDITIONAL CONSIDERATIONS

The temperature and humidity at the structure can potentially have an effect on the load test results. Most diagnostic load tests can be performed in a short timeframe; typically long enough to apply the load, allow the load to stabilize, and remove the load. For these quick tests, changes in temperature or humidity would be small enough to reasonably consider as constant. For longer tests, it may be helpful to conduct the test overnight when the temperature remains relatively stable without fluctuations with the movement of the sun. For longer tests, it is recommended to also install temperature sensors and use the data to make temperature corrections to the data after the test is complete (Transportation Research Board 2019).

The Manual for Bridge Evaluation highlights several factors that may contribute to the load carrying capacity of the bridge under testing levels which may not be present at higher loads. These factors include: unintended continuity due to seized, corroded, or frozen supports, unintended composite action, stiffness contribution from secondary or non-structural members. An effort should be made to attempt to identify if any of these play a significant role in the behaviour of the structure during the load test. These beneficial factors should be excluded from the analysis if the results will be used to extrapolate the behaviour at higher evaluation loads (American Association of State Highway and Transportation Officials 2011). An example of these unintended factors is composite action in decks that are not composite with the supporting beams. Often there is a bond between the deck concrete and the top of the girder that will contribute to some degree of composite action at lower load levels. At higher loads however, the bond limit

is exceeded and the composite action is lost, resulting is a sudden increase in the stress in the girders (CSA Group 2019b).

3 PROPOSED LOAD TESTING PROCEDURE FOR CONCRETE SLAB BRIDGES IN NEW BRUNSWICK

3.1 INTRODUCTION

In the Province of New Brunswick, some of the most common types of bridges are the reinforced concrete slab and rigid frame. Several of these structures do not contain transverse shear reinforcement in the deck. This is largely due to the fact that when these were originally designed the shear design methods in use at the time allowed a greater capacity from the concrete compared with today's methods. As a result, the concrete was considered to adequately resist the loads without the need for additional transverse reinforcement (American Concrete Institute 1985). As expected, these structures typically rate very poorly during an analytical load rating in accordance with Section 14 of S6. However, when inspected these same structures often do not show any significant signs of shear distress. Evidently there is some additional reserve capacity in the slab. This may be from concrete capacity or unknown load dispersion through the slab. This situation puts the New Brunswick Department of Transportation and Infrastructure (NBDTI) in a difficult position where these bridges are shown to perform well in service but have no analytical proof to validate it. This becomes an even larger problem when there is a request to permit an overweight vehicle on these structures. Past performances suggest that the structure should be able to carry the load, however, an analysis will show that it cannot. It is suggested that a diagnostic load test on these slab-type bridges be performed to refine the analysis and gain a better understanding of the structure's behaviour.

Where there are several of these types of structures around the province, a standardized method to load test these structures would help facilitate the implementation of load testing. This report aims to present a standardized process that NBDTI can follow in testing their structures. While this report is tailored for concrete slab-type bridges, the general methodology would be applicable to any type of bridge.

The following procedure is broken down into five main stages: Initial Investigation, Preliminary Analysis, Planning, Load Testing, and Refined Analysis. Figure 20 below outlines the main steps to be taken; detailed information about each step is included in the following sections. This process is presented generically and could easily be adapted to other structure types as well.

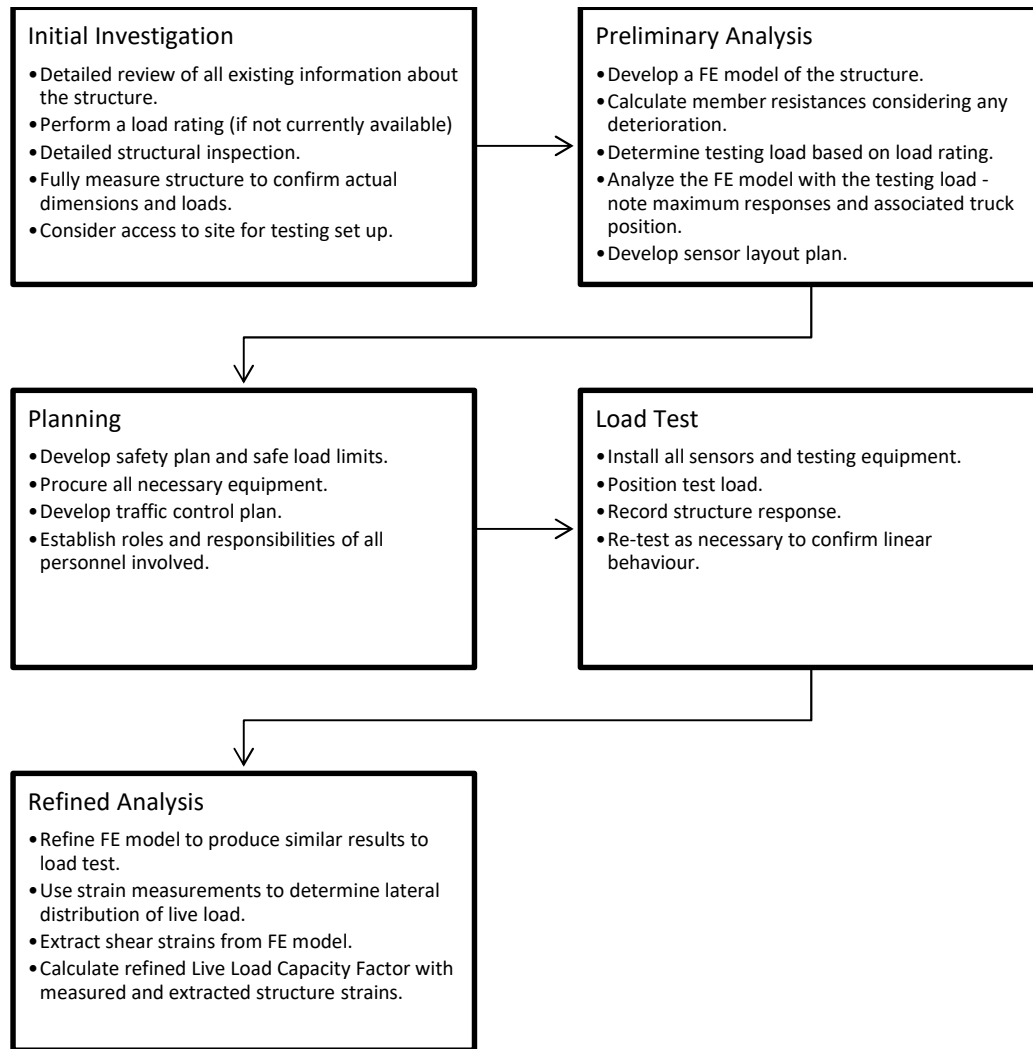


Figure 20 – Load Testing Procedure Flow-Chart

3.2 INITIAL INVESTIGATION

To begin, the structure must be investigated to collect all of the relevant data about it.

This information will be a mix of design and existing details that will be later used to better understand the structure and how it behaves.

A detailed review of any existing documents on the bridge should be performed. This includes the original design drawings and specifications, any previous inspection reports, any testing performed on the structure, and the most recent load rating calculations. The review should take note of all of the information that will be needed to fully model the structure as well as calculate resistances. This is a good opportunity for identifying missing information that could be collected during a site visit.

A detailed structural investigation should be performed on the bridge by a competent inspector trained in structural condition assessment of bridges. The inspector should take note of the existing condition of the bridge, noting any deterioration that may affect the behaviour of the structure. The bridge should be fully inspected and compared with the record or design drawings. Any discrepancies between the actual conditions and design drawings should be noted. Examples of potential differences may be different asphalt thickness, different member dimensions due to past rehabilitations, or different support conditions. Cracking of the concrete should also be documented to be monitored and compared with post-test conditions.

Frequently on slab-type bridges across the province, additional earth fill is added on top of the slab in an effort to improve the grading condition of the road. In addition to the fill, the travel surface likely has also been re-paved several times and may have several layers

of asphalt on top. This additional dead load on the structure has a significant effect on the load carrying capacity of the structure and must be documented thoroughly.

If some information remains unknown after the site visit, additional investigations may need to be performed. This could include taking core samples of the concrete to determine the actual concrete properties, or ground penetrating radar testing to locate and assess the embedded reinforcing.

The site visit should also take note of access at the bridge site. Items to consider are: a suitable area for personnel to set up and monitor the test, access to install sensors and equipment for the load test, potential power supply for the testing equipment, potential impacts on the travelling public, proximity of utilities, just to name a few.

3.3 PRELIMINARY ANALYSIS

At this stage, it is assumed that a load rating has already been performed identifying the bridge as a potential candidate for load testing. The preliminary analysis is to gain a better understanding of the bridge behaviour and plan out how to best carry out the load test. The initial load rating could be revisited to investigate the sensitivity of the load rating results to the effect of the dynamic load allowance (DLA). If a reduction in the DLA is greatly beneficial to the rating of the structure, it may be worthwhile to investigate performing a dynamic load test. For the purposes of this procedure, it is assumed that a dynamic load test will not be included.

The first step would be to take the information gathered during the initial investigation and develop an accurate finite element (FE) model of the actual structure. This is a critical step where the model will be used now to better understand the bridge behaviour

as well as after the load test to refine the analysis. How the model is created will depend on the structure, as well as engineering judgement and good modelling practices. Once the model is built, it is suggested that it be loaded with the CL-625-ONT evaluation vehicles and the results used to refine the existing load rating. Evaluation with the FE model may indicate that the structure has more capacity than initially thought during the initial load rating and eliminate the need for a full load test. Aborting the load test in favour of using the FE model for a refined load rating should only be done if the model is fully validated and the engineer is confident that it produces reasonable results.

If the structure resistances have not been calculated in a previous load rating, or if the structure inspection revealed deterioration that needs to be accounted for, they should be determined. It is important to know if the structure is flexure-critical or shear-critical and to plan the load test accordingly. As part of the preliminary calculations, Transportation Research Board (2019) and Lantsoght (2019a) suggest calculating the structure capacities without load and resistance factors when investigating the limits of the load test.

However, CSA Group (2019a) requires the resistance strains to be factored when calculating the live load capacity factor based on load test measurements. Lantsoght (2019a) recommends that the following properties be determined:

- Ultimate bending moment resistance based on average material properties
- Moment-curvature diagram based on average material properties
- Maximum applied load from vehicle or test tandem that causes yielding of the tension reinforcement
- Maximum applied load from vehicle or test tandem that causes flexural failure
- Shear resistance based on average material properties
- Maximum applied load from vehicle or test tandem that causes shear failure
- Punching shear resistance based on average material properties
- Maximum applied load from vehicle or test tandem that causes a punching shear failure

- Load-displacement diagram based on average material properties, which can be used for field test verification
- Initiation of the first cracks based on the cracking moment

The next step is to determine the load to be used for the load test. The structure is likely being load tested as a result of receiving a load rating factor less than one. As an initial starting point, the testing load can be based on the posted load limit resulting from the load rating. Load ratings in New Brunswick are performed using the CL-625-ONT evaluation vehicles. These three axle configurations are an idealized representation of large highway truck traffic. The axle configurations and loads are shown in Figure 21 below.

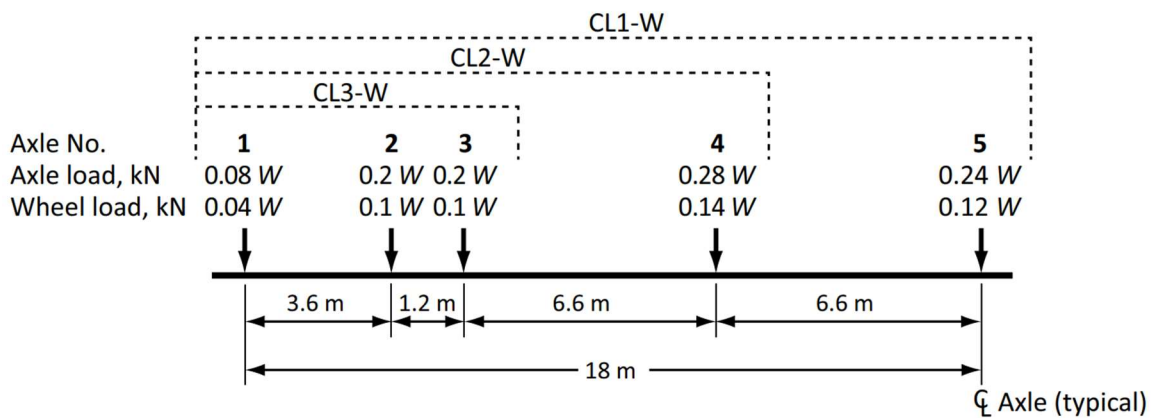


Figure 21 – CL1, CL2, and CL3 Truck Configurations – CSA Group (2019a)

The CL3 evaluation vehicle reasonably resembles a typical NBDTI tandem maintenance vehicle (Figure 22); as such, it is recommended that the NBDTI tandem maintenance vehicles be used to load the structure. These trucks are able to increase their gross weight to the target load with the addition of sand or earth fill typically readily available at NBDTI maintenance depots. Based on measurements taken of actual NBDTI maintenance vehicles and the “Heavy Truck Weight and Dimension Limits for

Interprovincial Operations in Canada”, an average weight distribution between axles for this type of vehicle is: 30% on the front axle, 35% on each of the two tandem axles.

While the actual truck used in the load test will need to be measured, a typical wheel layout for preliminary calculations can be seen in Figure 22 below.

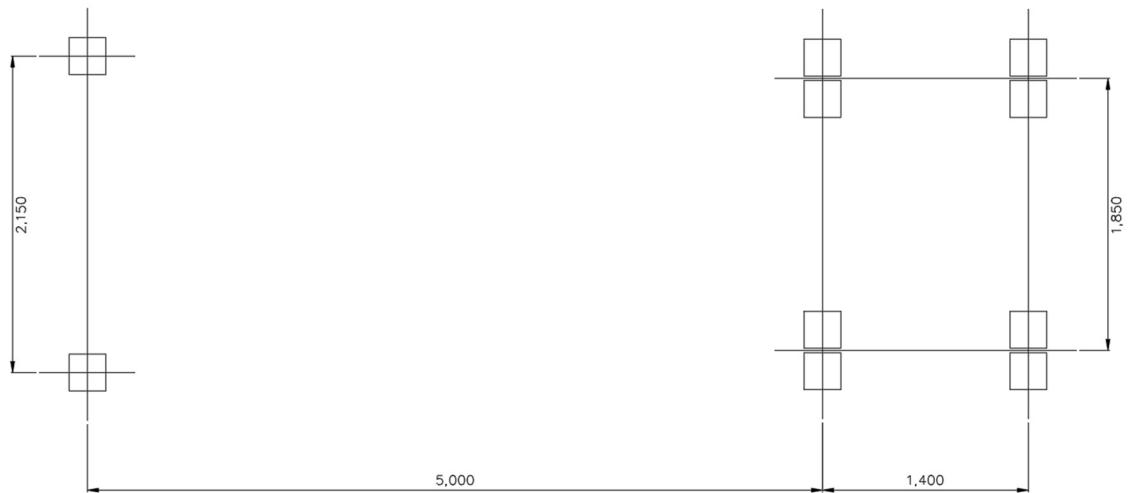


Figure 22 – Typical test truck wheel layout

Now that the model is built and the test load is known, a moving load analysis should be performed to identify the truck placement that will generate the greatest load response. Lantsoght (2019a) recommends that when applying the truck load to a slab model, the individual tire loads should be distributed through the asphalt and top half of slab at 45 degrees so that the load is applied at the centroid of the slab. The location of the greatest load response as well as the truck location that caused it should be recorded. Keeping in mind that the greatest response in the model may occur in a location that may not be accessible to measure, such as the max shear strain occurring at the centre of a column support. In addition to the location of the response, the magnitude of the response should also be noted as this will indicate the sensor range that will be required during the load

test. This is also a good time to compare the anticipated stresses in the structure during loading with the cracking stress of the concrete. In an effort to not subject the structure to non-linear behaviour, it is desirable to keep the stresses in the slab less than the cracking stress (Transportation Research Board 2019). It may become a delicate balance determining a testing load that will generate a large enough response to reliably register on the sensors yet be low enough to not cause cracking or non-linear behaviour. Finally, a sensor plan will need to be established for the structure. The sensor plan will need to be custom developed for the particular bridge being tested. Based on the preliminary analysis, the anticipated strains in the structure will determine the type and range of the sensors to be used in the load test. According to Lantsoght (2019a), the sensor plan should consist of the following items:

- Type, position, and range of all sensors that will be applied;
- Additional sensors with type and range of which the position will be applied in the field (e.g., for monitoring crack width in concrete bridges);
- Number, type, and range of backup sensors that need to be taken to the field;
- The required elements for data acquisition and real-time monitoring: data logger (and list identifying which sensor is connected to which canal), amplifier, analog-to-digital converter, and data visualization software;
- The properties of the different data acquisition and visualization elements, including the sampling rates;
- A list with calibration values of all sensors and the date of the most recent calibration;
- Details of sensor mounting for contact sensors;
- Wiring details for wired sensors.

Sensors will need to be installed at all locations where measurements are needed. This includes measurements required to model the structure response as well as measurements required for monitoring the safety of the test.

Direct measurement of shear stresses can be very difficult to observe in the field. Shear strains are at a maximum inside the slab and reduce to zero at the top and bottom surfaces. This leaves the outer side-face of the slab as the only viable location to measure shear strains on a reinforced concrete slab (Transportation Research Board 2019). Unfortunately, shear strains at the outside face of a slab are typically very low, if not negligible. To evaluate for shear, the shear response is taken from a finite element model that has been calibrated to produce the same flexural strains measured during a load test. For refined analysis and calibration of the FE model, strains measured on the bottom and top of the slab, combined with deflection measurements are typically sufficient (Lantsoght 2019a).

3.4 PLANNING

The success of a load test will greatly depend on the degree of planning and preparation that goes into it. All aspects of the test will need to be considered at some level. Each bridge will have particular challenges to address yet much of it will be common to all bridges.

The number one priority of any load test should be safety. This includes the safety of those involved in the testing, safety of the travelling public, and safety of the structure itself. While some of this can be mitigated with actions taken on site, most will be the result of an accurate preliminary analysis of the structure to establish safe loading limits and identify when to stop the test early if necessary.

Prior to the load test, all of the equipment necessary for the execution of the test must be identified and procured. This includes all sensors and data acquisition systems,

equipment required for installation of the sensors, temporary power sources, and the test truck. Arrangements should also be made to have the truck weighed and loaded to the specified weight.

Consideration of the traffic at the structure will also need to be addressed. If the structure spans over a roadway, the traffic on the road below should be stopped until the test is complete, and the test load has been removed. As well, depending on the size of the bridge being tested, the traffic on the bridge may also need to be stopped until the test is completed. In both cases, traffic control, signage, and potential detours must be considered. Any modifications to the regular traffic flow should be implemented according to the New Brunswick Work Area Traffic Control Manual (WATCM).

Finally, all of the personnel that will be needed in relation to the test should be identified. Communication on site during the load test is critical to a successful test. Everyone should be aware of their particular roles and responsibilities to avoid any confusion during the test.

3.5 LOAD TESTING

Now that a preliminary analysis has identified the test load, load placement, and sensor plan, the load test can be carried out. The load test will consist of three phases: installation of testing equipment, placement of test truck load, and recording of structure response. While a diagnostic load test can typically be performed in a relatively short period of time, it may be desirable to perform the test in the evening when temperature fluctuations are at a minimum.

The first step is to install all of the required sensors for the test. Depending on the bridge layout and site conditions, a variety of access equipment may be required to install the equipment. Unless a permanent power source is available nearby, a generator should be on site to power the testing equipment. Prior to application of the test load, the sensors should all be calibrated and tested to ensure that the output is reliable.

The testing load and load placement has been determined during the preliminary analysis. Once the test truck has been loaded to the required weight, the exact axle configuration and wheel loads should be measured and documented. If necessary, traffic on the bridge should be temporarily stopped while personnel measure out and mark on the bridge deck the exact truck placements. Once all preparations have been made, traffic should be stopped, and the test truck directed into position to begin the test.

The test truck should be left in place until the various sensor responses stabilize.

According to Piotr Olaszek and Joan R. Casas (Lantsoght 2019a), it is recommended to allow the load to remain in place a minimum of 30 minutes . Alternatively, provided that the sample rate is 5 minutes or less, the response can be considered stable if the change in response relative to the previous reading is less than 0.2%. It is highly recommended that the sensor outputs be monitored while the test is being carried out. The responses should be continuously compared with the anticipated response calculated from the preliminary analysis. If possible, it is helpful to have data acquisition equipment that can also provide a graphical display of the incoming data. Some systems allow for superimposing the calculated responses over the test data, allowing for simple visualization of the data.

Active monitoring of the data also allows the test engineer to identify issues with the

sensor output. The Primer on Bridge Load Testing (Transportation Research Board 2019) suggests watching for any of the following sensor outputs:

- A sudden shift in the measurements caused by an impact to the sensor and/or wiring;
- A constant response value known as a “flat line: (i.e., if the sensor has reached the limitations of its measurement range);
- Displacement and strain readings in the opposite of the expected direction caused by reversed signal wires;
- A sudden offset or response that does not return to zero could indicate that the sensor has detached from the structure;
- A noisy signal (poor signal-to-noise- ration) due to missing or improperly configured amplifiers, crosstalk, or other electrical phenomena; and
- Drifting sensors due to grounding issues.

A diagnostic load test should keep the structure in the linear elastic range during its response. Typically, the loads used in a diagnostic test are low enough that this should not be an issue. In order to test for linearity, the test should be repeated several times to compare the response. The structure can be said to behave linearly if the response reasonably be replicated with the same load applied at the same location. Alternatively, linearity can also be shown by loading with half of the original weight and observing half of the original response (Dong et al. 2020). If the response cannot be replicated, or if the output does not return to pre-loaded state, this may be an indication of non-linear behaviour. Although, in most cases, it is likely due to sensor issues. Regardless, the cause of the irregular response must be investigated and a decision must be made on whether or not to continue the test (Transportation Research Board 2019).

While the data should be monitored in real-time during the load test, some amount of post processing may need to be performed after the test is complete. Depending on how long the test lasted and the environmental conditions during the test, strain measurements may

need to be adjusted to consider the temperature effects. If deflection measurements were taken at the supports, the mid-span deflections should be modified to reflect this. In some cases, it may be beneficial to apply a filter to noisy data to allow for cleaner data to work with. Once the data has been corrected and cleaned, the data should be organized and plotted to allow for further use. Peak values should be identified along with their respective load position. Time histories for each sensor output with regard to time and load position can also be useful (Transportation Research Board 2019).

3.6 REFINED ANALYSIS

Once the load test results have been successfully collected and validated, a refined analysis of the structure will be performed. This will include refining the FE model from the preliminary analysis with load test results, determining actual lateral load distribution, and calculating the updated LLCF.

The FE model will need to be refined to produce very similar responses to those measured during the load test before it can be used in the refined load rating. To do so, the bridge properties (such as moment of inertia and modulus of elasticity) in the model are adjusted so that the model behaves as closely as possible to the actual bridge.

Lantsoght (2019a) suggests comparing the measured deflections with the deflections from the refined model to determine when the model has been refined sufficiently. They suggest an acceptable match between the model and the real bridge is when the maximum deflections are within 15% for reinforced concrete bridges.

With the data collected from the strain gauges positioned in a transverse line at mid-span, the actual lateral distribution of live load can be determined. Eq. 7 below from S6 (CSA

Group 2019a) gives the minimum value for truck fraction for a unit width strip of deck with a distribution factor of 1.05. As the live load is spread transversely across the deck, the amount of live load that is carried by a unit width longitudinal strip of deck depends on the transverse stiffness of the slab. The truck fraction parameter gives the portion of live load carried by a strip of deck.

$$F_T = 1.05 \frac{nR_L}{B_e} \quad \text{Eq. 7}$$

Where:

F_T	Truck fraction
n	Number of trucks
R_L	Multiple lane reduction factor
B_e	Effective width of deck

Where the load test was performed with a single test truck, n and R_L become 1.0. Taking the maximum strain divided by the average strain across the five sensors, gives the distribution factor for the structure. The final equation for the truck fraction is shown in Eq. 8 below.

$$F_T = \frac{\varepsilon_{max}}{\varepsilon_{AVG}} \times \frac{1}{B_e} \quad \text{Eq. 8}$$

Where:

F_T	Truck fraction
ε_{max}	Maximum strain due to live load
ε_{AVG}	Average strain due to live load
B_e	Effective width of deck

Once the FE model has been refined to produce reasonably similar results as the load test it can be used to extract shear strains in the slab that could not be directly measured in the field. The FE model should be loaded with the same test truck loads as used during the actual load test and placed in a manner as to produce the greatest shear response in the slab. In accordance with Clause 8.9.4.1 of S6 (CSA Group 2019a), the critical section for shear is located a distance d from the face of support which is where the shear strains should be extracted from the model. These strains, along with the flexural strains from the load test, will be used in the refined load rating of the structure. When load effects are taken from FE models of concrete bridges, it is recommended to take the average load effect over 3 m in the transverse direction for flexure and a distance $4d$ for shear, where d is the depth to longitudinal reinforcing (Lantsoght 2019a). Pacoste et al. (2012) suggests that the width for shear should be limited to $5d$. However, Lantsoght et al. (2013, 2015) has found that for concentrated load close to a support, the effective width for shear is more accurately represented by the French load spreading method where the load is spread at 45° from the back side of the load towards the support as shown in Figure 23 below. Reissen and Hegger (2012) concluded that spreading loads at less than 45° underestimates the slab capacity in shear.

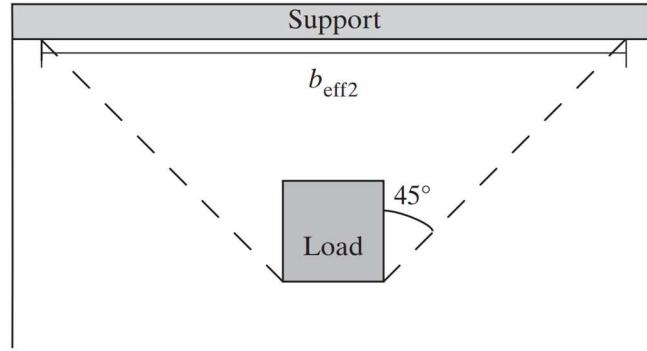


Figure 23 - Distribution of shear forces near a support – Lantsoght (2013)

The simplified 2D beam model from the preliminary load rating analysis can be re-used to determine the live load effects due to the test truck and the CL625-ONT evaluation vehicle. The load effects should be taken at the same location as the sensors to allow for direct use of the load test measurements. The previously calculated distribution factor can now be applied to the load effects to give the load effect on a per metre strip basis. If a dynamic test was chosen, the revised DLA can also be applied at this time.

Depending on the results of the load test, calculation of the refined LLCF will either follow the method for Situation 1 or Situation 2 from S6.1. In most cases however, Situation 1 will be the governing case. To calculate the LLCF with Situation 1, strains ϵ_e and ϵ_{DL} are calculated using their respective code clauses, $\epsilon_{test\ truck\ measure}$ is taken directly from the load test measurements, and the load effects for the test truck and CL625-ONT evaluation vehicle are taken from the simplified 2D beam model. Combining all of these variables in Eq. 1, will give the refined LLCF for the structure.

3.7 SUMMARY

The proposed process outlined above condenses to five main steps: Initial Investigation, Preliminary Analysis, Planning, Load Testing, and Refined Analysis. It is proposed that this procedure could help guide NBDTI in load testing concrete slab bridges in New Brunswick. In addition to gaining a better understanding of the bridge, the results from a load test could be used to update the estimate of the remaining service life of the structure. Load testing an aging bridge could uncover additional capacity and delay the need for additional maintenance to extend the life of the structure. The effects on the life cycle of the bridge would need to be assessed on a case-by-case basis as testing may reveal a significant increase in capacity, minimal change in capacity, or even a reduction in capacity.

4 CASE STUDY – LOAD TESTING AND REFINED LOAD RATING OF THE ST. LEONARD C.N.R. OVERHEAD NORTH BRIDGE

4.1 INTRODUCTION

A case study was performed on a reinforced concrete slab bridge in New Brunswick. This structure is known as St. Leonard C.N.R. Overhead North and is located approximately two kilometers north of St. Leonard on the westbound lane of Route 2 of the TransCanada Highway as shown in Figure 24. The bridge is a 3-span continuous reinforced concrete slab as shown in Figure 26, Figure 27, and Figure 27 below.

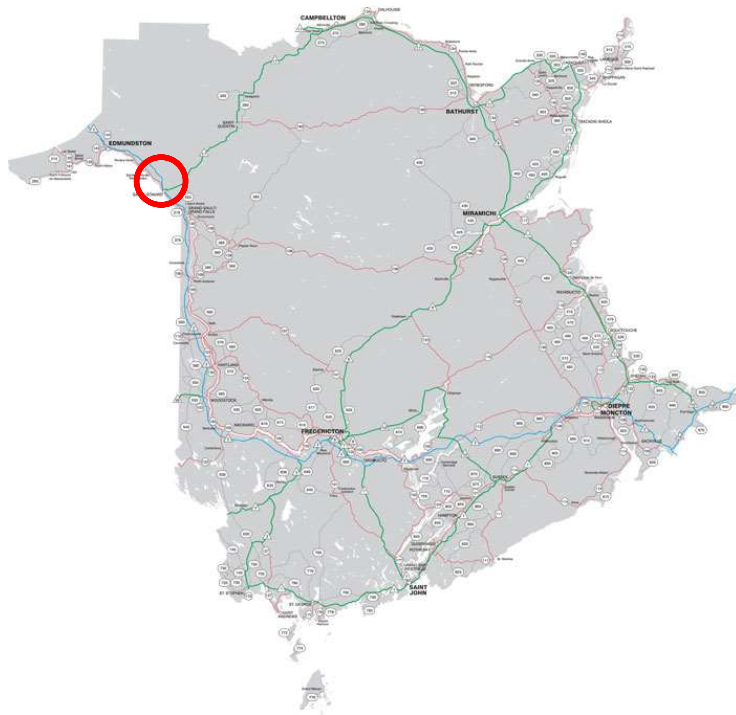


Figure 24 – Bridge Location¹⁰

¹⁰ <https://www.gnb.ca/0113/maps/nb-hwypmap-e.asp>



Figure 25 - Elevation view of St. Leonard C.N.R. Overhead (North)

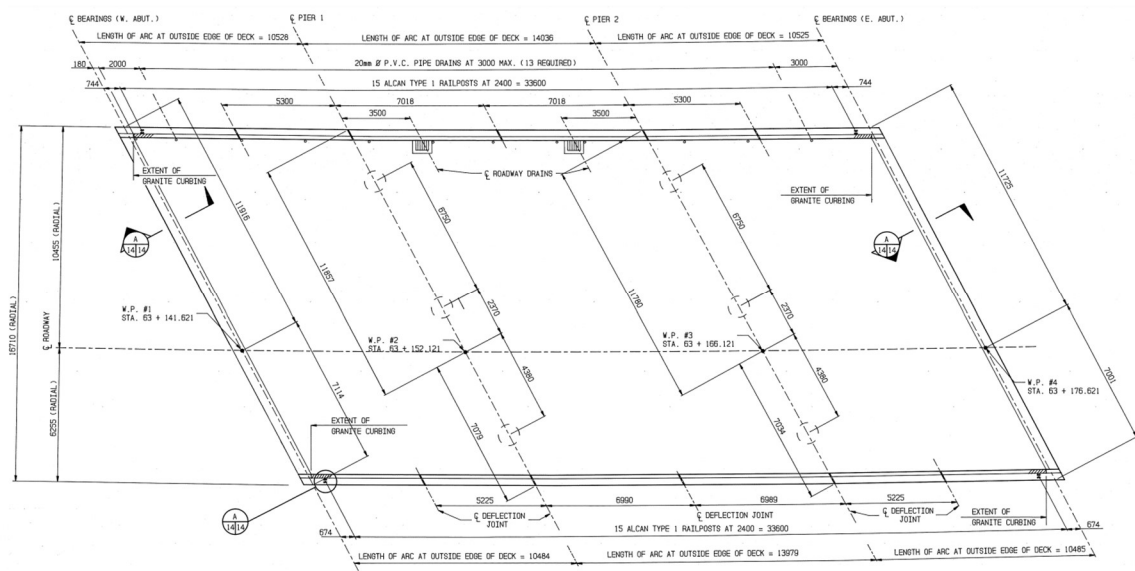


Figure 26 – Plan of St. Leonard C.N.R. Overhead (North)

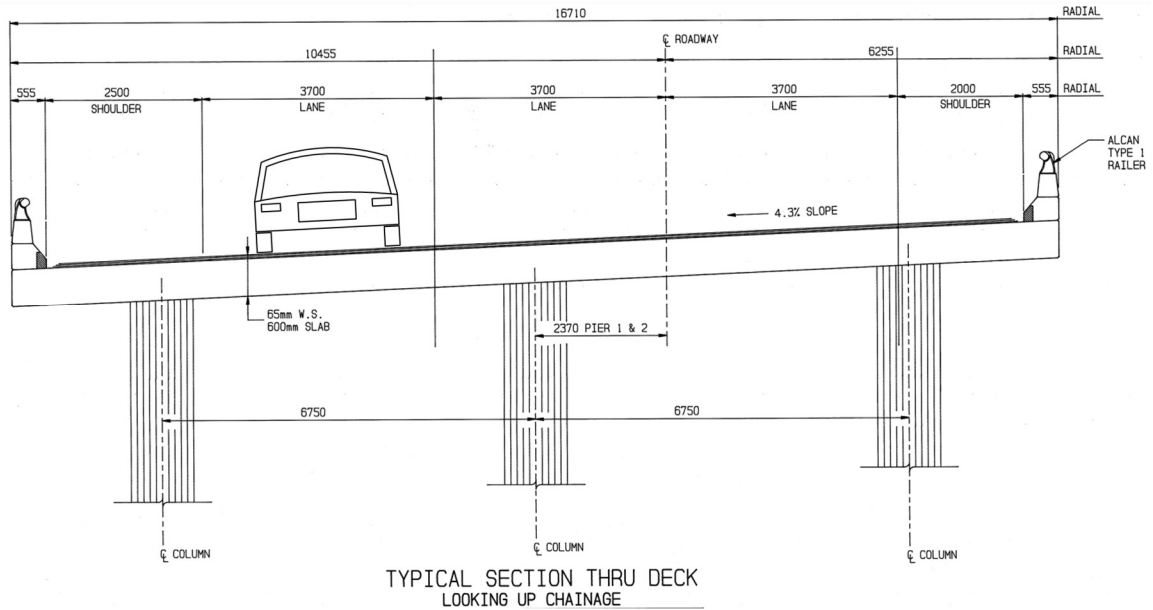


Figure 27 – Typical Section of St. Leonard C.N.R. Overhead (North)

This structure had been load rated previously and was found to rate poorly in shear (MacDonald 2020). Similar to other concrete slab structures, this structure does not exhibit signs of distress due to loading. This bridge was selected to perform a static load test on and use the results to calculate a more refined shear capacity.

In lieu of performing an actual real-world load test on this bridge, a detailed finite element (FE) model of the bridge was developed as accurately as possible in the finite element software LUSAS to represent the bridge. For the purposes of this case study, the FE model of the bridge will be referenced to as *The Bridge* and all references to it will be as if it were a real load test. The stresses and strains were extracted from the bridge model at the locations where a sensor would have been installed during an actual load test.

4.2 INITIAL LOAD RATING

As an initial analysis, the slab was analyzed through a Beam Strip analysis with distribution factors determined by the Simplified Method as found in Section 5.6 of S6-19. The slab was found to have distribution factor of 0.249 for flexure in the slab and 0.520 for shear in the slab. The analysis was carried out using the structural analysis software SFRAME to model a 1 m wide strip of the bridge. The model considered 30 MPa concrete for both slab and columns. The model was checked using both uncracked and cracked section properties for the slab as a lower and upper bound. Taking the inflection point under dead load as the transition point, the cracked section properties were different for positive bending areas and negative bending areas. The cracked moment of inertia of the slab was calculated through Eq. 9 and Eq. 10 (Pillai et al. 1999).

$$I_{cr} = \frac{b(kd)^3}{3} + (nA_s)(d - kd)^2 \quad \text{Eq. 9}$$

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n \quad \text{Eq. 10}$$

Where:

I_{cr}	Cracked moment of inertia
b	Section width
d	Section depth
n	Modular ratio
A_s	Area of reinforcing steel
k	Coefficient for effective depth of concrete
ρ	Ratio of reinforcing to concrete

The section properties used for the model are shown in Table 2. Note that the same area is used for both cracked and uncracked section properties because SFRAME does not use

the section area for stiffness calculations, rather it uses it for calculating the self-weight of the member.

	Uncracked		Cracked			
			Positive Bending		Negative Bending	
	Moment of Inertia (mm ⁴)	Area (mm ²)	Moment of Inertia (mm ⁴)	Area (mm ²)	Moment of Inertia (mm ⁴)	Area (mm ²)
Slab	18.0×10^9	600×10^3	3.33×10^9	600×10^3	4.94×10^9	600×10^3
Upper Column	8.81×10^9	141×10^3	8.81×10^9	141×10^3	8.81×10^9	141×10^3
Lower Column	70.3×10^9	795×10^3	70.3×10^9	795×10^3	70.3×10^9	795×10^3

Table 2 – SFRAME Model Properties

Using Section 8 of S6-19, the minimum shear resistance of the slab at the pier was determined to be 354.4 kN, the positive flexural resistance was 413.3 kN-m, and the negative flexural resistance was 651.5 kN-m. From analysis, the maximum factored shear force at the pier was determined to be 412 kN (Figure 28) for uncracked condition and 368 kN (Figure 29) for cracked condition. The maximum factored moment in the slab at mid-span was 435 kN-m (Figure 30) for uncracked condition and 351 kN-m (Figure 31) for cracked condition.

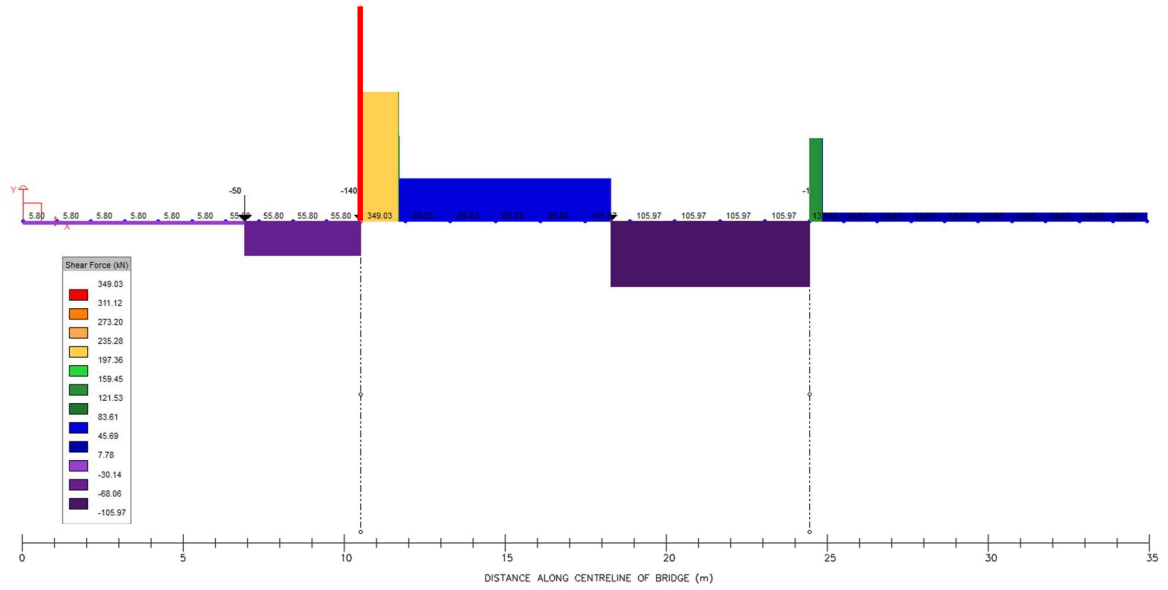


Figure 28 – Maximum Shear Force (CL1 loading, Uncracked)

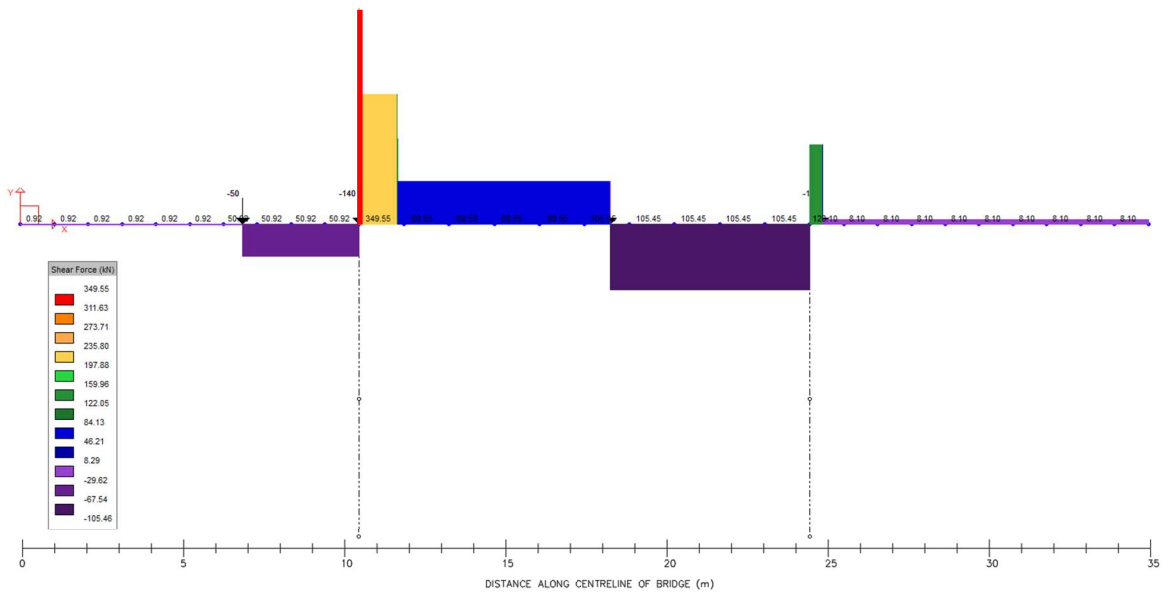


Figure 29 – Maximum Shear Force (CL1 loading, Cracked)

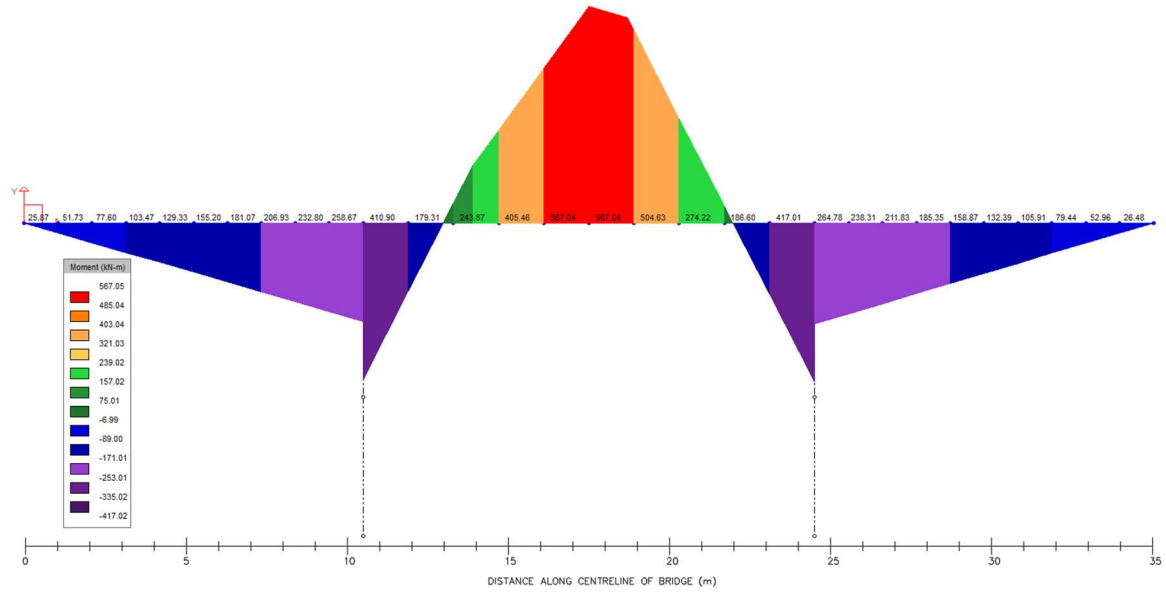


Figure 30 – Maximum Moment (CL3 Loading, Uncracked)

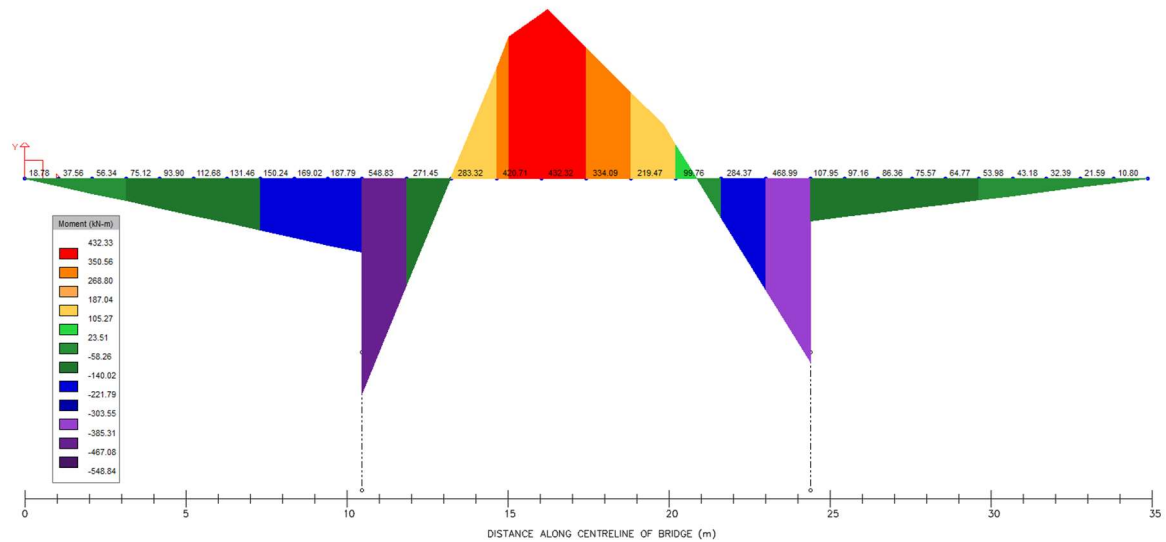


Figure 31 – Maximum Moment (CL3 Loading, Cracked)

The Live Load Capacity Factors (LLCF) for the slab were determined in accordance with Section 14 of S6-19. The minimum LLCF for the structure are shown in Table 3. A LLCF less than 1.0 indicates that the combination of permanent and live loads exceeds the factored capacity of the structure. In this situation, a LLCF of 0.88 implies that the structure can only support 88% of the evaluation vehicle before the capacity is exceeded.

Uncracked		Cracked	
LLCF _M	LLCF _V	LLCF _M	LLCF _V
0.92	0.96	1.29	0.88

Table 3 – Live Load Capacity Factor Results

In accordance with Section 14.17 of S6, the corresponding posted loads were determined to be 22 tonnes for the CL3 evaluation vehicle. Where the test truck is most similar to the axle configuration of the CL3 evaluation vehicle, the gross weight of the test truck was limited to 22 tonnes.

4.3 HYPOTHETICAL LOAD TEST

A hypothetical load test was carried out on the structure through the use of finite element modelling software in place of testing the actual structure. A preliminary model was built, and the test load was applied to determine where the sensors should be placed during the load test. The load test was then carried out on a detailed finite element model to observe the structure response.

4.3.1 LOAD AND LOAD PLACEMENT

The load for the load test was considered to be a typical NBDTI maintenance tandem truck. The truck was assumed to be loaded with sand to increase the loaded axle weights to the target truck load of 22 tonnes. The test truck axle configuration can be seen in Figure 32 and a typical maintenance truck is shown in Figure 33 below.



Figure 32 – Test truck axle configuration



Figure 33 – Typical NBDTI maintenance vehicle

The test truck was to be placed on *The Bridge* to produce the greatest load effect in the slab. The position of the test truck to achieve this was determined using the simplified 2D SFRAME beam model. The truck position is detailed in Table 4 below for each load effect.

Load Effect	Rear Axle Stationing from End of Bridge
Max M+ in Span 1	2.8 m
Max M+ in Span 2	16.1 m
Max M- at Pier	13.8 m
Max V at Abutment	0.0 m
Max V at Pier	10.5 m

Table 4 – Test truck placement for maximum load effect

4.3.2 PRELIMINARY ANALYSIS

The next phase of the evaluation involved building a simplified FE model of the bridge. The finite element software LUSAS was used to build and run the model. For this structure, the horizontal curve was ignored and considered to be straight. The radius of the horizontal curve was large enough to allow the use of the simplified method, so assuming the bridge to be straight was considered acceptable. Due to the horizontal curve, the actual skew of the abutments and supports varied along the length of the bridge. For the simplified model, the skew angles were averaged, and the same 28° skew angle was used at all supports. The elastomeric bearings and column supports were all simplified to be pin supports. The model was built using quadrilateral thick shell elements and 30 MPa concrete properties. Two versions of the model were built: one considering uncracked section properties and one considering cracked section properties. Once the models were completed, they were validated by running under its own self weight and comparing the vertical reactions with hand calculations. The simplified model can be seen in Figure 34 and Figure 35 below.

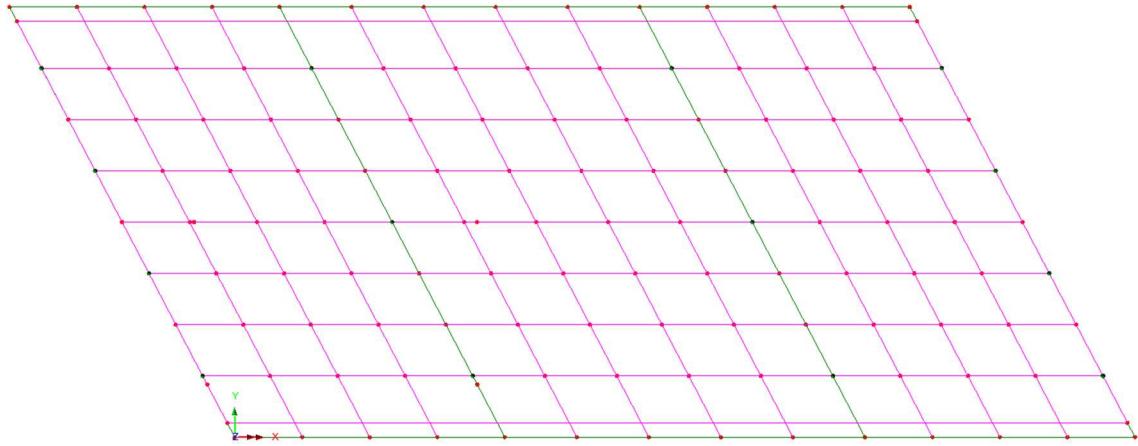


Figure 34 – Plan of simplified FE model

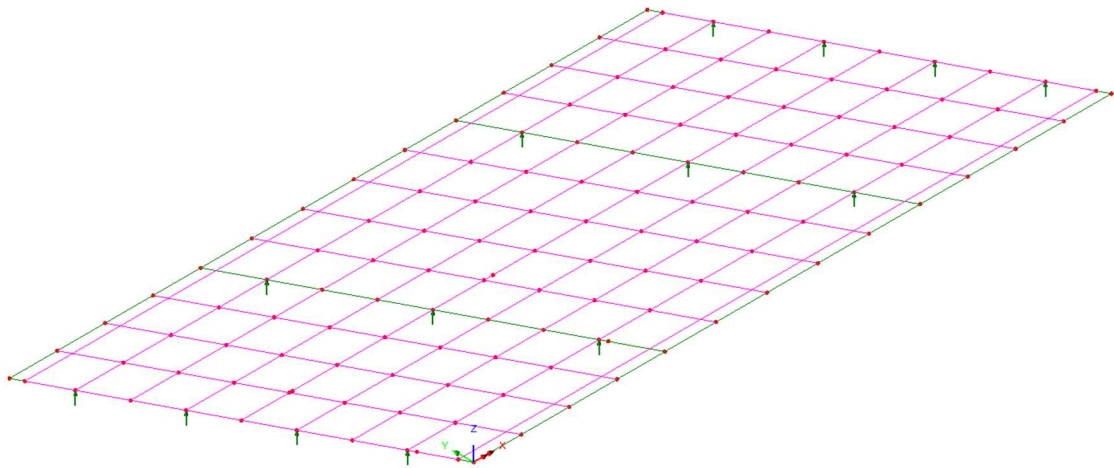


Figure 35 – Isometric view of simplified FE model

The simplified FE model of the bridge was loaded with the test truck at the above indicated locations at the centreline of the bridge. The truck wheel loads were applied as point loads to be conservative. The model response under live loads indicated the range of strains that would be expected during the actual load test; allowing appropriate sensors to be selected. For this preliminary test, the strains ranged from 12 to 19 $\mu\epsilon$ under uncracked conditions and 57 to 68 $\mu\epsilon$ under cracked conditions depending on the truck location. The shear strains will be extracted from the refined FE model after calibration

with load test results. Figure 36, Figure 37, and Figure 38 show the strain contours for each flexural loading scenario under uncracked condition while Figure 39, Figure 40, and Figure 41 show the strain contours for each flexural loading scenario under cracked condition.

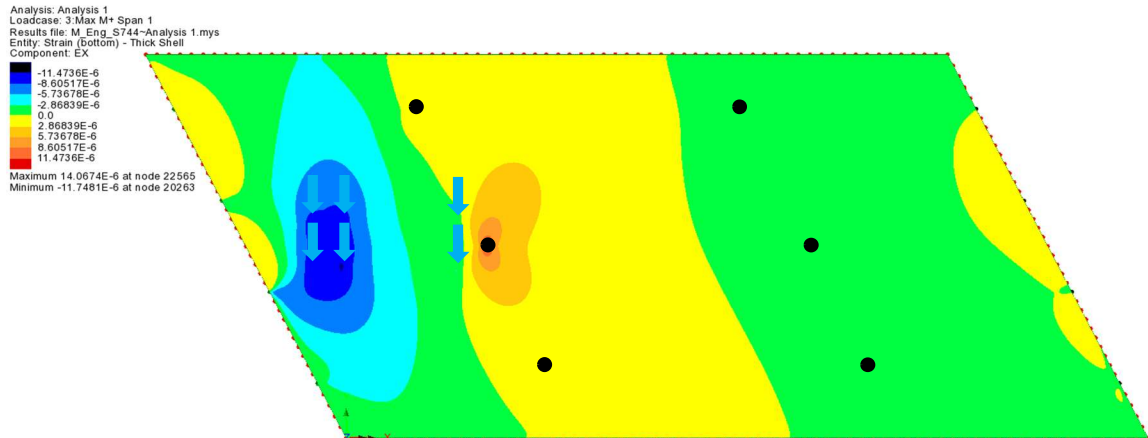


Figure 36 – Max longitudinal strain on bottom of slab in Span 1 (Uncracked)

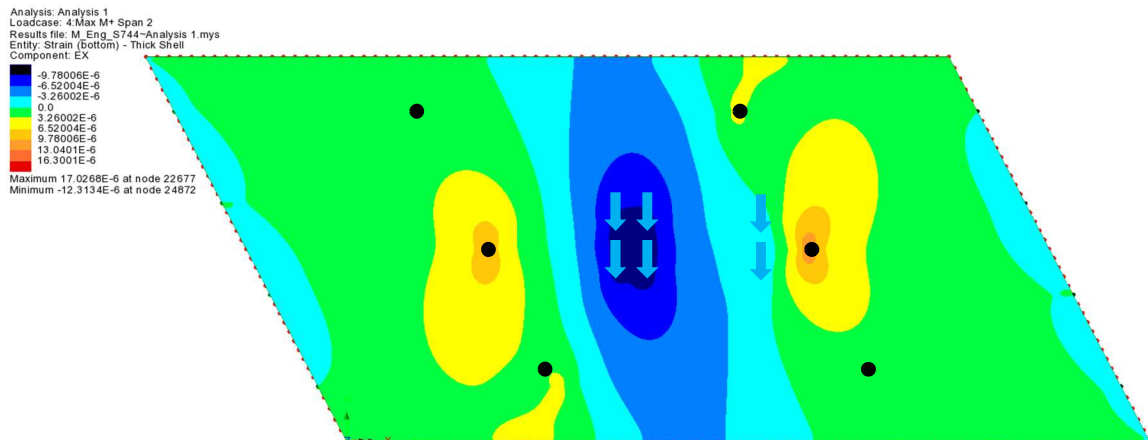


Figure 37 – Max longitudinal strain on bottom of slab in Span 2 (Uncracked)

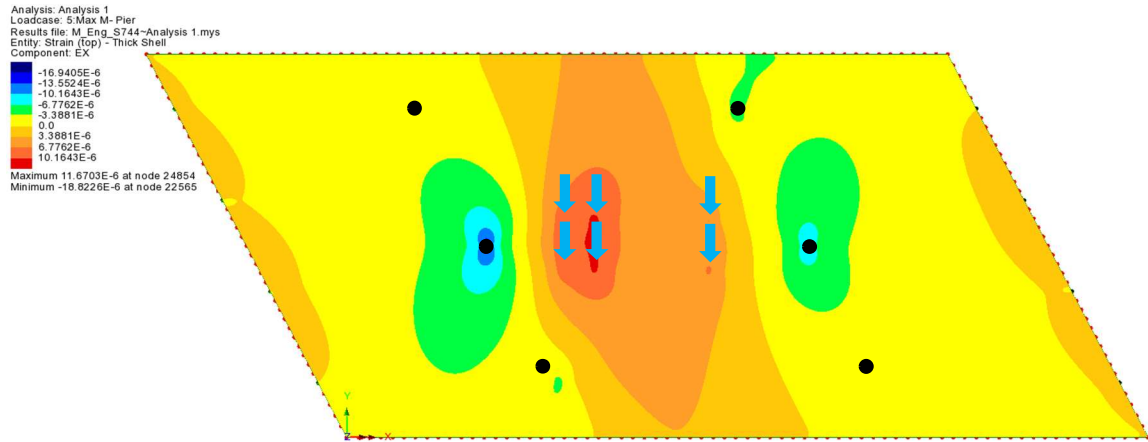


Figure 38 – Max longitudinal strain on top of slab over Pier 1 (Uncracked)

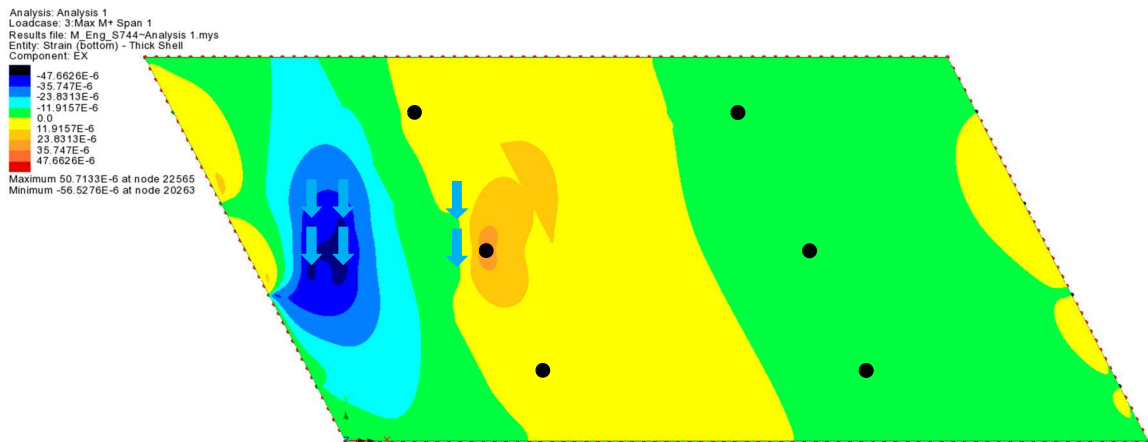


Figure 39 – Max longitudinal strain on bottom of slab in Span 1 (Cracked)

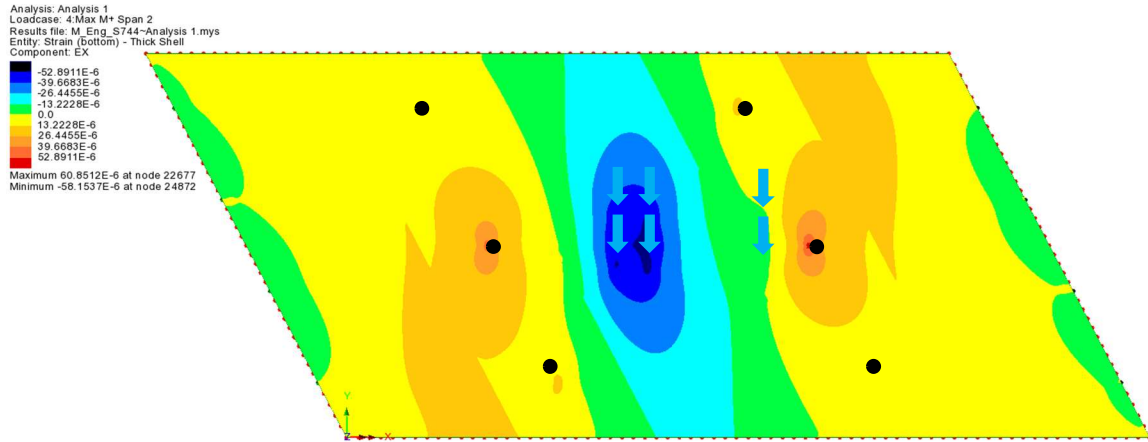


Figure 40 – Max longitudinal strain on bottom of slab in Span 2 (Cracked)

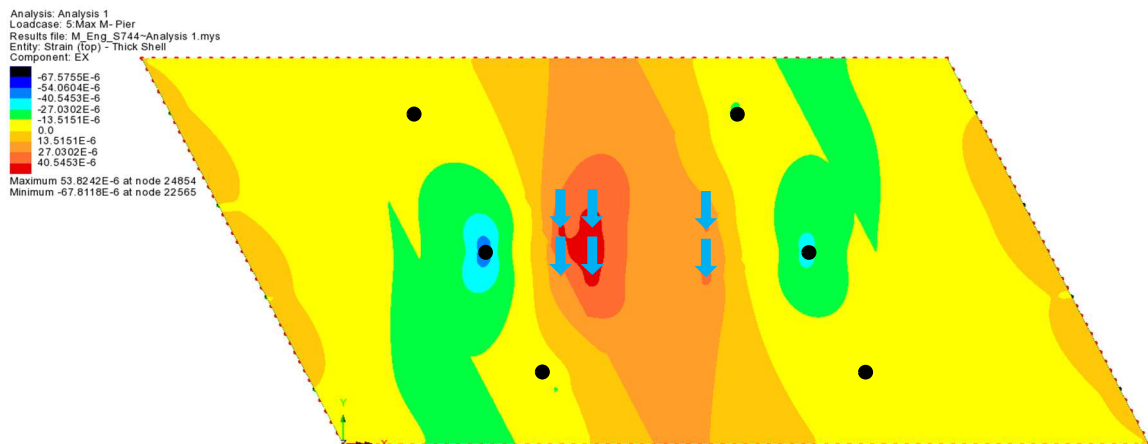


Figure 41 – Max longitudinal strain on top of slab over Pier 1 (Cracked)

Being a skewed slab bridge, the transverse strains (across the deck width) in the slab were also investigated. Figure 42, Figure 43, and Figure 44 show the transverse strains in the slab under cracked condition. The longitudinal strains were found to be 20-80% larger than the transverse strains suggesting that the critical direction for sensors to measure is the longitudinal strains.

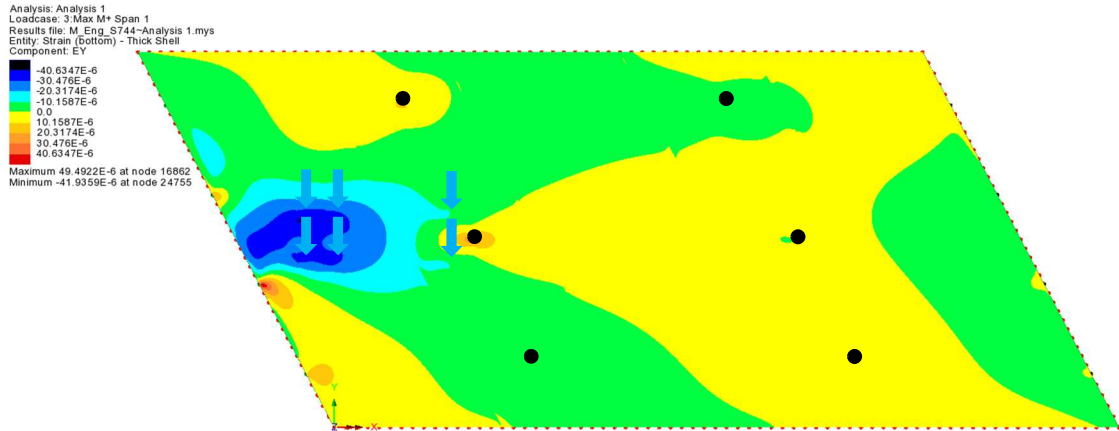


Figure 42 - Max transverse strain on bottom of slab in Span 1 (Cracked)

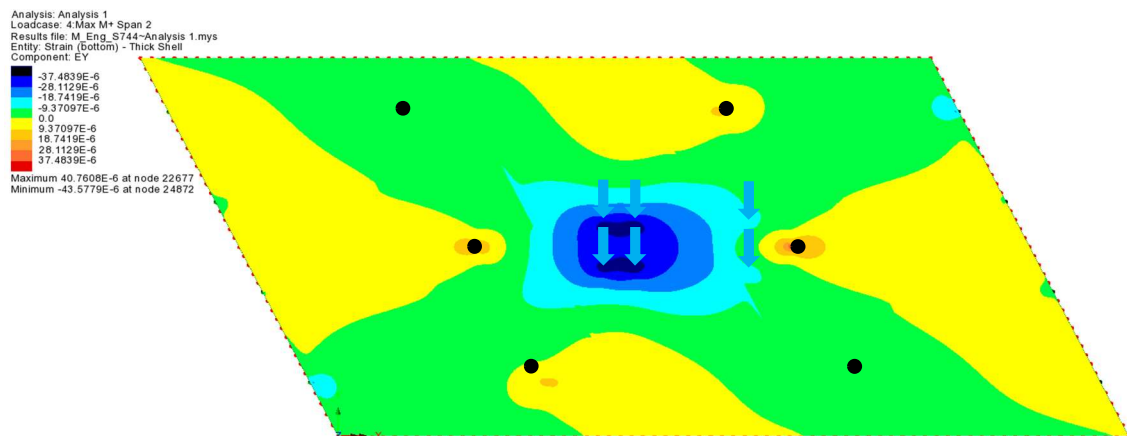


Figure 43 - Max transverse strain on bottom of slab in Span 2 (Cracked)

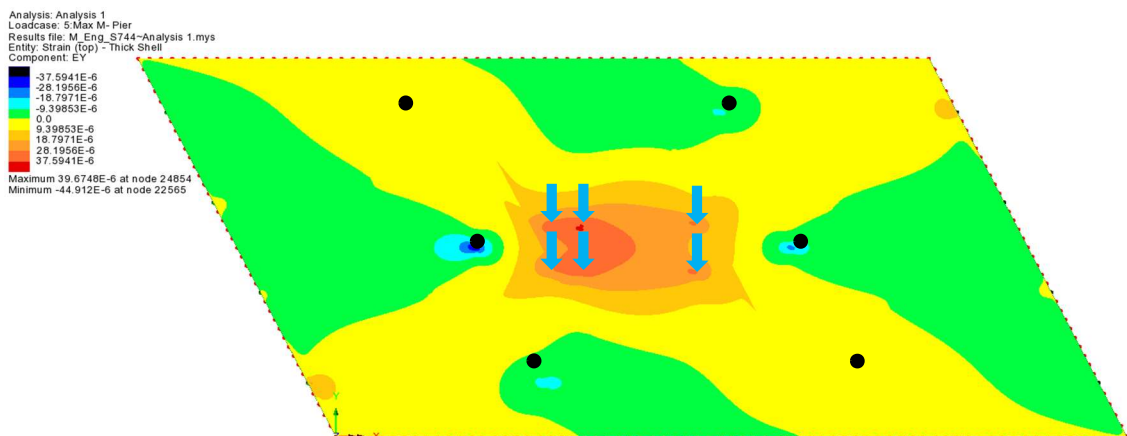
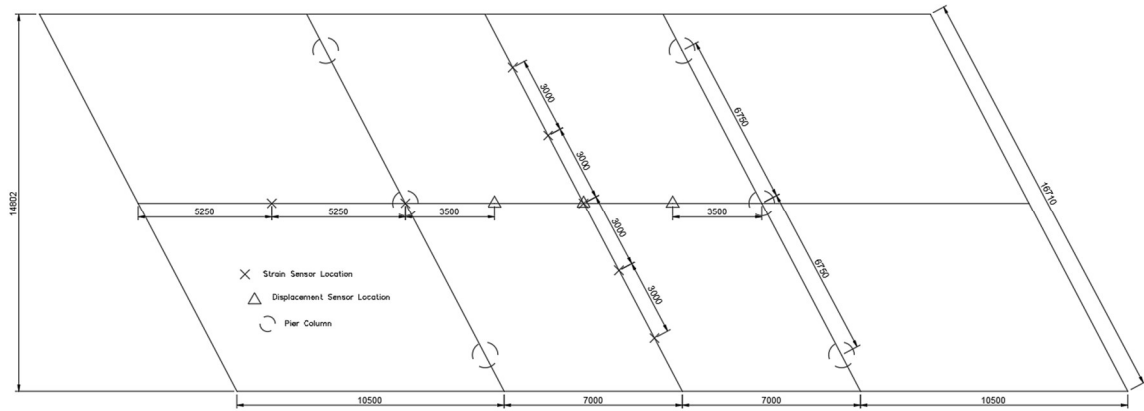


Figure 44 - Max transverse strain on top of slab over Pier 1 (Cracked)

It was also at this stage where the model was run to see if any of the strains would exceed the cracking strain of the slab concrete. The maximum longitudinal strain observed was $68 \mu\epsilon$ which is lower than the cracking strain of $77 \mu\epsilon$ for the 30 MPa concrete in the slab. The maximum transverse strain observed was $40 \mu\epsilon$. This indicated that the slab was unlikely to experience non-linear behaviour during the load test.

4.3.3 SENSOR LAYOUT

Where this load test was hypothetically carried out, no actual sensors were applied to the bridge. If it was a real load test, demountable strain gauges and LVDT sensors for deflections would have been used. For illustrative purposes, the location where the FE model outputs were taken from are being considered 'sensor locations'. Where the revised load rating will be focusing on the critical areas of the structure, the sensors were placed in select locations to capture the maximum load effects. To capture the maximum positive flexural strains, the gauges were placed at the mid-span on the bottom face of the slab at the centreline. To capture the maximum negative flexural strains, a gauge was placed on the top face of the slab above the pier. To install this sensor, a portion of the asphalt was hypothetically removed to install the sensor directly on the slab concrete. To measure the transverse distribution of live load, five strain gauges were positioned in a line along the centre of Span 2. The centre gauge was placed at the centreline while the two side gauges were offset 3 m and 6 m respectively. In addition, the displacement sensors were placed at quarter points of Span 2. This allowed for validation of the deflected shape of the slab during calibration of the FE model. The sensor layout plan is shown in Figure 45 below.



4.3.4 SIMULATED LOAD TEST

The refined FE model of *The Bridge* was also built in LUSAS. The model was built to consider the actual layout and dimensions per the design drawings, including horizontal curve and skewed supports. It also included fully modelled piers and barriers. The bearings at the abutments were modelled as springs with the same stiffnesses as the actual elastomeric bearings. With the bridge being 30 years old and on a main section of the Trans-Canada Highway, it was assumed that the bridge may have experienced sufficient loading to have previously introduced cracks in the slab. As such, the model used cracked section properties in the slab. The refined model representing *The Bridge* can be seen in Figure 46 and Figure 47.

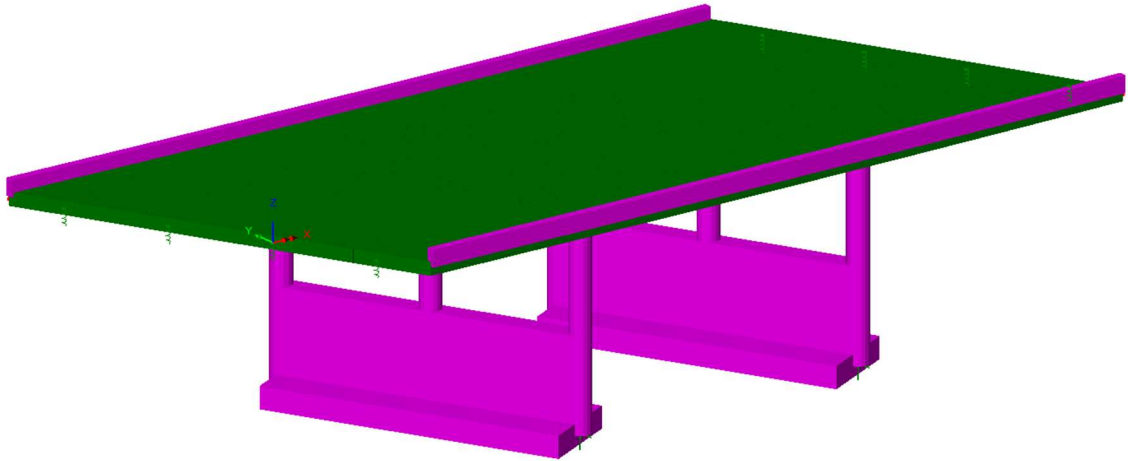


Figure 46 – 3D view of *The Bridge* model

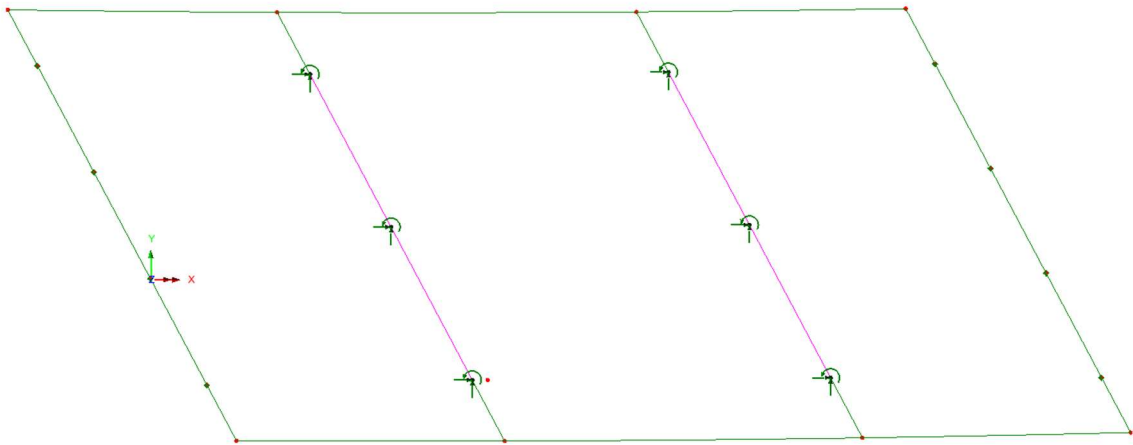


Figure 47 – Plan view of *The Bridge* model

The Bridge was loaded with the test truck at locations as indicated above. The maximum strains recorded from the sensors are presented in Table 5 below.

Location	Measured Strains
Max M+ in Span 1	39.05 $\mu\epsilon$
Max M+ in Span 2	62.59 $\mu\epsilon$
Max M- at Pier	30.64 $\mu\epsilon$
3.5 m off centreline	12.7 $\mu\epsilon$
6.5 m off centreline	42.76 $\mu\epsilon$
9.5 m at centreline	62.59 $\mu\epsilon$
12.5 m off centreline	29.65 $\mu\epsilon$
15.5 m off centreline	12.09 $\mu\epsilon$

Table 5 – Sensor Measurements

4.4 REFINED LOAD RATING

The strains measured during the load test were then used to calibrate the simplified FE model to reflect the actual response. The refined model used the cracked section properties of the slab. To calibrate the model, the model properties were manipulated until the deflection in the model fell within 15% of the deflections observed in the load test as recommended by Lantsoght (2019a). Similar to the model of *The Bridge*, the refined model also used the actual stiffnesses for the abutment bearings. The equivalent spring stiffness of the columns were calculated and used as spring supports at the piers. For the lateral spring stiffness of the columns, a single column was modelled in SFRAME and a unit load was applied to the top of the column. Dividing the unit load by

the deflection in the model gave the lateral spring stiffness. For vertical spring stiffness, Eq. 11 was used.

$$\delta = \frac{PL}{AE} \text{ combined with } k = \frac{P}{\delta} \text{ to give } k = \frac{AE}{L} \quad \text{Eq. 11}$$

The modulus of elasticity of the slab was reduced by 4.4% for positive bending areas and 9.7% in negative bending areas. These changes resulted in the following deflections in Table 6.

Location	<i>The Bridge</i>	Refined Model	% Difference
Span 1	1.58 mm	1.40 mm	11.4%
Span 2	2.29 mm	2.27 mm	0.9%

Table 6 – Comparison of deflections between load test and refined model

Once the calibrated model was complete, the shear strains at the supports were extracted. The shear strains were all taken at the distance $d_v = 485$ mm from the face of the reaction area in accordance with 8.9.4.1 of S6-19. Where the primary bending occurs longitudinally along the deck, the shear strains were taken in the transverse vertical plane YZ as shown in Figure 48. Strains were also investigated in the XZ plane but were found to not govern. The maximum shear strain was found to be $30.50 \mu\epsilon$ at the abutment while the maximum shear strain at the pier was found to be $28.02 \mu\epsilon$. These are identified by the red line in Figures 49 through 52.

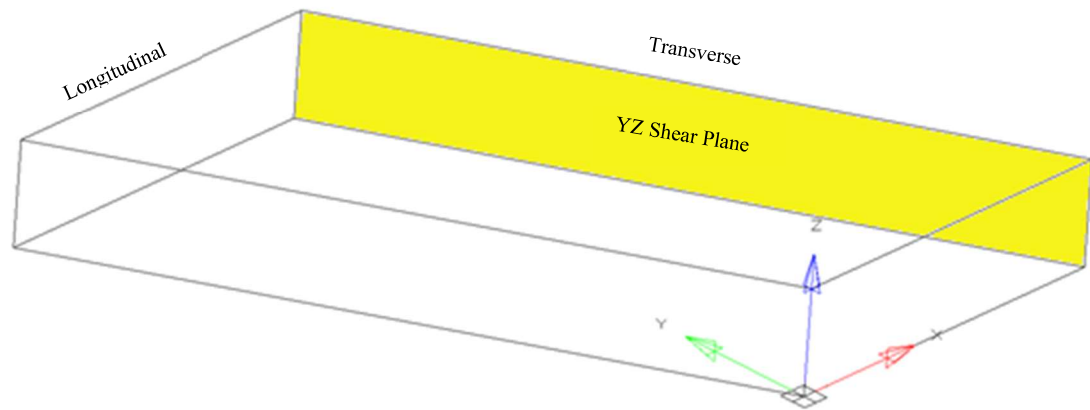


Figure 48 – YZ Shear Plane

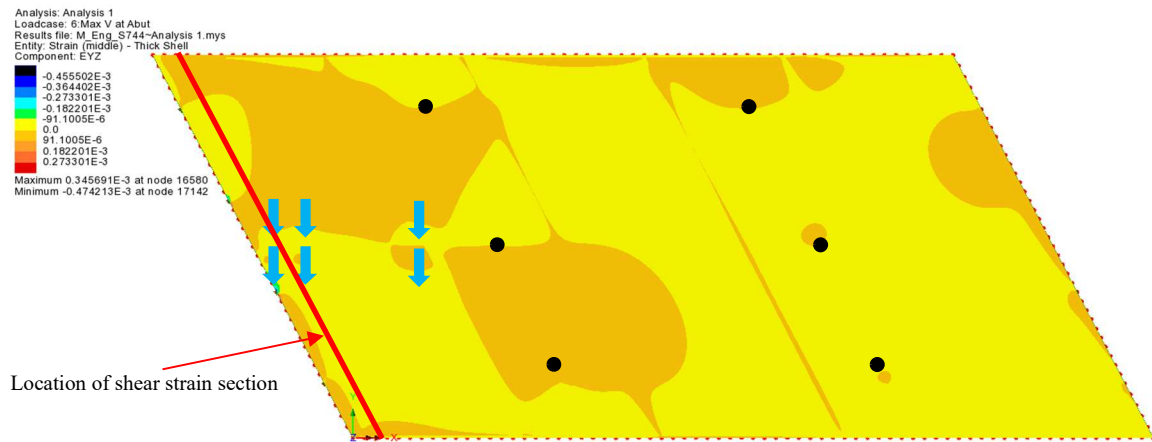


Figure 49 – Shear strains at abutment

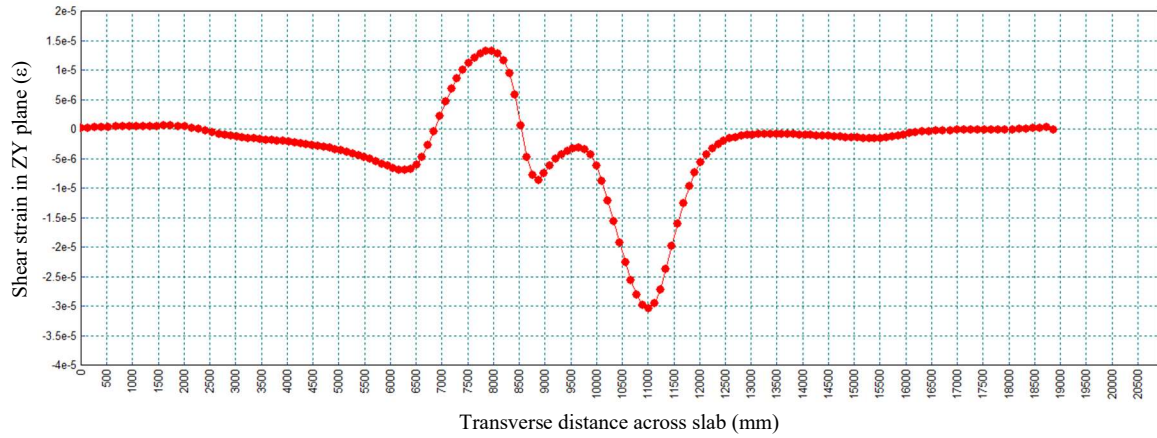


Figure 50 – Cross-section of shear strain taken at d_v from face of bearings at abutment

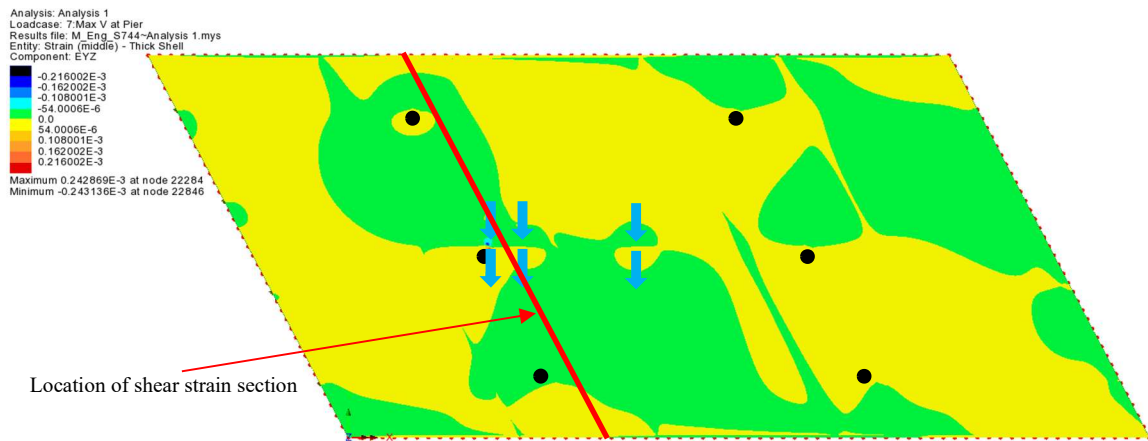


Figure 51 – Shear strains at pier

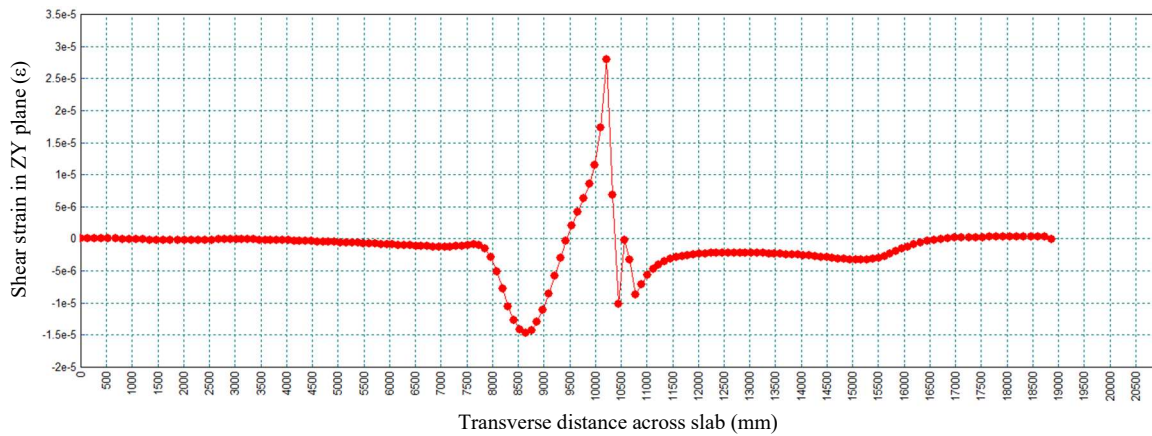


Figure 52 - Cross-section of shear strain taken at d_v from face of pier

The shear strains were averaged over an effective width of slab in accordance with the French load spreading method. It was assumed that the load was spread from the tandem axles at 45 degrees to the support face as shown in Figure 53. The effective width was found to be 10.78 m giving an average strain of $3.50 \mu\epsilon$ at the abutment and $2.16 \mu\epsilon$ at the pier. Similarly, the measured flexural strains were averaged over a 3 m width of slab. The final adjusted strain measurements are shown in Table 7. With all of the measured strains from the test truck and the FE model collected, the refined load rating was able to take place.

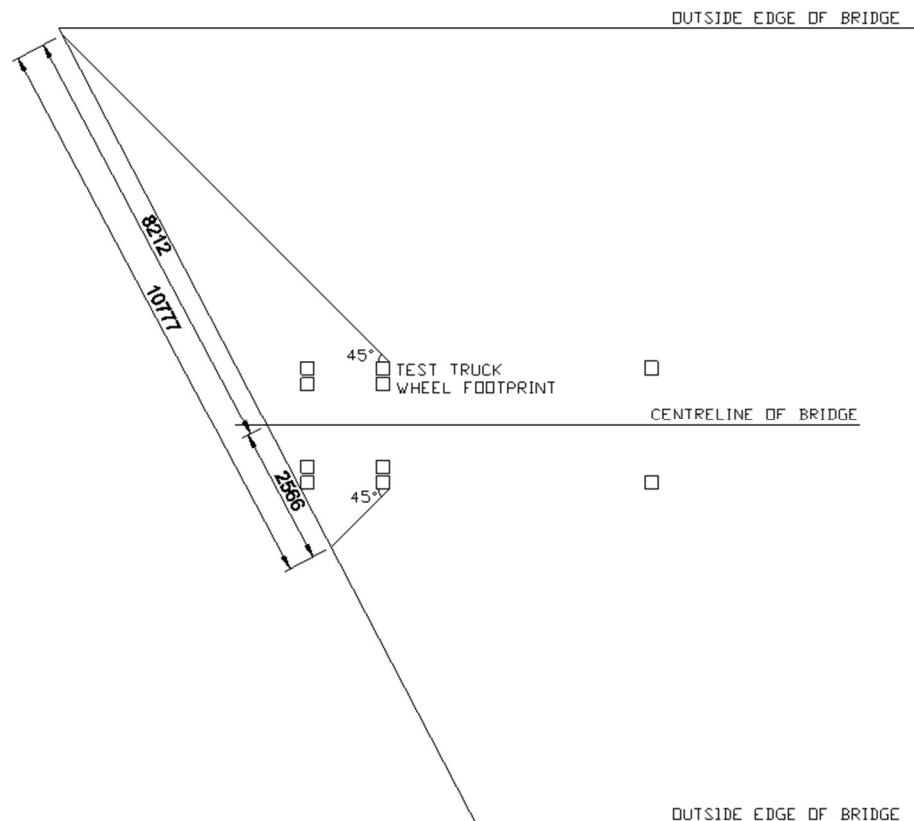


Figure 53 – Effective slab width per French load spreading method

Location	Adjusted Strains
Max M+ in Span 1	13.02 $\mu\epsilon$
Max M+ in Span 2	20.86 $\mu\epsilon$
Max M- at Pier	10.21 $\mu\epsilon$
Max V at Abutment	3.50 $\mu\epsilon$
Max V at Pier	2.16 $\mu\epsilon$

Table 7 – Adjusted Strains

Using the strains measured at 3 m intervals in the transverse direction at the centre of Span 2, the actual transverse load distribution was calculated to be 0.117 using Eq. 12 below:

$$Distribution\ Factor = \frac{Max\ Strain}{Average\ Strain} \times \frac{1}{Deck\ Width} \quad \text{Eq. 12}$$

Section 14.16.4.2 of S6-19 provides direction on how to use results from a load test to calculate a refined LLCF. During the load test, it was identified that the stress in the slab did not exceed the cracking stress under the test truck loading. As such, the test results can be categorized as Situation 1 as defined in S6.1-19 and the associated Eq. 13 can be applied.

“Situation 1: When a load test is stopped, for example, when all available test trucks and loading blocks have been used or the limitations of the testing equipment capacity have been reached, but with no apparent damage, deformation, or change in the linear behaviour of the bridge. In this case, the maximum applied test load may be less than the CHBDC design load. The actual

bridge capacity conforming to the CHBDC design load may be determined by extrapolating the test results subject to approval.”

$$F = \frac{(\epsilon_e - \epsilon_{DL}) \frac{L_{test\ truck\ theor}}{\epsilon_{test\ truck\ measure}}}{L_{CHBD\ truck\ theor} \alpha_L (1 + l)} = \frac{(\epsilon_e - \epsilon_{DL})}{\epsilon_{test\ truck\ measure}} \times \frac{L_{test\ truck\ theor}}{L_{CHBD\ truck\ theor} \alpha_L (1 + l)} \quad \text{Eq. 13}$$

The resistance strains were calculated based on a 1.0 m wide unit strip of deck. Where the shear resistance is dependent on the longitudinal strain in the slab, the shear resistance strains were calculated individually for each evaluation vehicle at the abutment and pier support locations. The moment and shear resistances had been previously calculated for the simplified load rating. From these resistances, the flexural and shear stresses were calculated using Eq. 14 and Eq. 15 respectively based on the uncracked gross section properties of the slab section. In turn, the flexural and shear resistance strains were finally determined. Table 8 and Table 9 show the flexural and shear resistance strains.

$$\epsilon = \frac{M_r \times y}{I} / E \quad \text{Eq. 14}$$

$$\epsilon = \frac{V_r}{A} / E \quad \text{Eq. 15}$$

	Resistance (kN-m)	Width (mm)	Depth (mm)	Centroid (mm)	M.O.I. (mm ⁴)	Stress (MPa)	Modulus of Elasticity (MPa)	Resistance Strain $\mu\epsilon$
M+	413.3	1000	600	300	1.80E+10	6.888	28259	243.76
M-	651.5	1000	600	300	1.80E+10	10.858	28259	384.24

Table 8 – Flexural Resistance Strains

	Resistance (kN)	Width (mm)	Depth (mm)	Cross- Sectional Area (mm ²)	Stress (MPa)	Modulus of Elasticity (MPa)	Resistance Strain $\mu\epsilon$
V @ Abut CL1	347.8	1000	600	600000	0.580	28259	20.51
V @ Abut CL2	346.5	1000	600	600000	0.578	28259	20.44
V @ Abut CL3	341.9	1000	600	600000	0.570	28259	20.16
V @ Pier CL1	356.6	1000	600	600000	0.594	28259	21.03
V @ Pier CL2	357.8	1000	600	600000	0.596	28259	21.10
V @ Pier CL3	365	1000	600	600000	0.608	28259	21.53

Table 9 – Shear Resistance Strains

The dead load strains were derived from the flexural and shear forces found in the simplified 2D SFRAME model. Similar to the resistances, the forces were translated into stresses based on the gross section properties, then into their equivalent strains. These strains were taken from the 2D model at the corresponding locations to the strain gauge measurements on *The Bridge*. The strains resulting from dead load are shown in Table 10 and Table 11 below.

Sta.	Location	M _f (kN-m)	Width (mm)	Depth (mm)	Centroid (mm)	M.O.I. (mm ⁴)	Stress (MPa)	Modulus of Elasticity (MPa)	Dead Load Strain $\mu\epsilon$
5.25	M+ Span 1	121	1000	600	300	1.8E+10	2.018	28259	71.41
17.5	M+ Span 2	169	1000	600	300	1.8E+10	2.812	28259	99.51
10.5	M- Pier	283	1000	600	300	1.8E+10	4.710	28259	166.68

Table 10 – Dead Load Flexural Strains

Sta.	Location	V _f (kN)	Width (mm)	Depth (mm)	Cross- Sectional Area (mm ²)	Stress (MPa)	Modulus of Elasticity (MPa)	Dead Load Strain $\mu\epsilon$
0	V at dv from abut.	61	1000	600	600000	0.101	28259	3.57
10.5	V at dv from pier	110	1000	600	600000	0.183	28259	6.49

Table 11 – Dead Load Shear Strains

The load effects from the test truck and the CL1, CL2, and CL3 evaluation vehicles were all taken directly from the simplified 2D model at the corresponding locations of the strain gauges. The lateral load distribution factors calculated from the load test data were applied to the live load effects to reduce them to a corresponding unit strip of slab. The live load factors determined with the target reliability index during the simplified analysis were also applied to this refined evaluation. Where the load test consisted of static loading only, a refined dynamic load allowance for the structure was not available for the refined evaluation. As a result, the 1.25 dynamic load allowance per 14.9.1.7 in S6 was also used here. The live load effects from the 2D model are shown in Table 12 and Table 13.

Sta.		Moment per Truck (kN-m)	Distribution Factor	Moment per 1 m Strip (kN-m)
5.25	M-Test	236	0.117	27.59
	M-CL1	466	0.117	54.49
	M-CL2	466	0.117	54.49
	M-CL3	466	0.117	54.49
17.5	M-Test	241	0.117	28.18
	M-CL1	437	0.117	51.12
	M-CL2	449	0.117	52.51
	M-CL3	456	0.117	53.34
10.5	M-Test	248	0.117	28.98
	M-CL1	591	0.117	69.10
	M-CL2	572	0.117	66.96
	M-CL3	464	0.117	54.30

Table 12 – Flexural Live Load Effects

Sta.		Shear per Truck (kN)	Distribution Factor	Shear per 1 m Strip (kN)
0	V-Test	153	0.117	17.91
	V-CL1	308	0.117	36.07
	V-CL2	310	0.117	36.25
	V-CL3	321	0.117	37.52
10.5	V-Test	156	0.117	18.28
	V-CL1	291	0.117	34.02
	V-CL2	290	0.117	33.93
	V-CL3	278	0.117	32.58

Table 13 – Shear Live Load Effects

Applying the above values to Eq. 13, the refined LLCF values for the structure are shown in Table 14 and Table 15 below.

Comparing these results with those calculated with the simplified method resulted in an average 87% increase in shear rating at the pier and 6% at the abutment. The load test also resulted in a 65-153% increase in flexural rating in positive bending and a 29% increase in negative bending. This increase could be attributed to a couple different reasons. A more accurate distribution of live load led to a reduction in live load strains applied to a unit width of slab. The unit width of slab was treated as a beam of a unit width, when it is actually part of larger laterally continuous slab. This allows the surrounding slab to provide additional stiffness to the unit width strip that is otherwise ignored. The increased stiffness, in turn, leads to less deflection and less strain in the slab.

Sta.		Load Effects (kN-m)	ϵ_e	ϵ_{DL}	ϵ_{TEST}	α_L	DLA	LLCF	Initial Load Rating	% diff
Mid Span 1	M-Test	27.59	243.76	64.96	13.02					
	M-CL1	54.49	243.76	64.96	13.02	1.49	1.25	3.73	1.47	153%
	M-CL2	54.49	243.76	64.96	13.02	1.49	1.25	3.73	1.47	153%
	M-CL3	54.49	243.76	64.96	13.02	1.49	1.30	3.59	1.42	153%
Mid Span 2	M-Test	28.18	243.76	81.19	20.86					
	M-CL1	51.12	243.76	81.19	20.86	1.49	1.25	2.31	1.40	65%
	M-CL2	52.51	243.76	81.19	20.86	1.49	1.25	2.25	1.36	65%
	M-CL3	53.34	243.76	81.19	20.86	1.49	1.30	2.13	1.29	65%
At Pier	M-Test	28.98	384.24	179.57	10.21					
	M-CL1	69.10	384.24	179.57	10.21	1.49	1.25	4.51	3.49	29%
	M-CL2	66.96	384.24	179.57	10.21	1.49	1.25	4.66	3.60	29%
	M-CL3	54.30	384.24	179.57	10.21	1.49	1.30	5.52	4.27	29%

Table 14 – Refined Flexural Live Load Capacity Factors from Load Test

Sta.	at dv	Load Effects (kN)	ϵ_e	ϵ_{DL}	ϵ_{TEST}	α_L	DLA	LLCF	Initial Load Rating	% diff
At Abutment	V-Test	17.91								
	V-CL1	36.07	20.81	3.45	3.50	1.63	1.25	1.21	1.13	7%
	V-CL2	36.25	20.76	3.45	3.50	1.63	1.25	1.20	1.12	7%
	V-CL3	37.52	20.46	3.45	3.50	1.63	1.3	1.10	1.06	3%
At Pier	V-Test	18.28								
	V-CL1	34.02	20.12	6.61	2.16	1.63	1.25	1.65	0.88	86%
	V-CL2	33.93	20.35	6.61	2.16	1.63	1.25	1.68	0.90	87%
	V-CL3	32.58	21.09	6.61	2.16	1.63	1.3	1.77	0.95	87%

Table 15 - Refined Shear Live Load Capacity Factors from Load Test

4.5 CONCLUSION

The St. Leonard C.N.R. Overhead North bridge in St. Leonard New Brunswick was load rated through the Simplified Method in Section 5.6 of S6-19. This method resulted in a minimum LLCF value of 0.88 in shear. A load test was hypothetically carried out on the structure through the use of a detailed FE model. The results of this load test were used to further refine the load rating in accordance with S6-19. The refined load rating resulted in a revised minimum LLCF of 1.65 in shear. This corresponds to an average improvement in the load rating of 86% in shear for this particular structure.

5 SUMMARY

5.1 GENERAL CONCLUSIONS AND SUMMARY

Load testing provides a means to determine the capacity of bridges more accurately.

Proof load tests allow for determining the minimum safe loading of a structure while diagnostic load tests give a glimpse into the actual structure response, which can then be used to assess the bridge under different loading scenarios. Measuring the structure response allows for the inclusion of secondary stiffening components, potential alternate load paths, and actual load distributions that are excluded from traditional analysis methods.

A hypothetical load test was carried out on the St. Leonard N.N.R. Overhead (North) concrete slab bridge through the use of finite element software. This bridge was selected as it is representative of the types of bridges in New Brunswick which rate poorly in shear capacity yet show no signs of distress. Using the methods in the Canadian Highway Bridge Design Code, the results of a traditional load rating was compared with a load rating using the structure response observed during the load test. The load test resulted in an average 7% at the abutment and 87% at the pier increase in shear live load capacity factor for this structure. It seems that load testing reinforced concrete slab bridges in New Brunswick may provide the means to quantify the apparent additional shear capacity in these structures.

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APPENDIX A: PROPOSED TESTING PROCEDURE

The following is a summarized flowchart of the load testing procedure developed in the main body of the report. This procedure is tailored for static diagnostic load tests of reinforced concrete slab bridges tested in New Brunswick; however, the general procedure can be applied to most bridges.

1.0 Initial Investigation

An initial investigation is performed on the structure to gather as much information about the bridge as possible. The goal is to be able to make well informed decisions about how to perform the load test.

- A. Collect and review all available records for the bridge (record drawings, inspection reports, and photos).
- B. Identify any missing information.
- C. Identify critical areas to inspect.
- D. Perform an analytical load rating if one is not available.
- E. Perform a detailed structural inspection of the bridge.
- F. Make note of the current condition of the bridge and any deterioration that may affect the structure behaviour.
- G. Fully measure all aspects of the bridge. Compare with existing drawings if available.
- H. Note any additional load that the bridge is carrying such as additional layers of asphalt or earth fill.
- I. Perform any additional testing if necessary, such as concrete core samples.
- J. Make note of site conditions for conducting the load test.

2.0 Preliminary Analysis

A preliminary analysis is carried out to gain a better understanding of the bridge behaviour. The information gathered during the initial investigation is utilized in this step.

- A. Update member resistances based on observed conditions.
- B. Build a simplified FE model based on field measurements.
- C. Load the FE model with CL3 evaluation vehicle and extract maximum forces.

- D. Revise initial load rating with forces from FE model.
 - The revised load rating may reveal additional capacity previously unknown. The engineer may decide that continuing with the full load test is unnecessary.
- E. Use the load rating results to establish test vehicle load.
- F. Re-analyze FE model with test vehicle load noting the location and magnitude of strains.
- G. Develop a sensor plan to capture maximum responses for refined analysis, monitor response for non-linear behavior, and establish lateral distribution of live load.

3.0 Planning

Careful preparation of each aspect of the load test is critical to a safe and successful test.

- A. Procure all necessary equipment and sensors to run the test.
- B. Make arrangements for borrowing/renting the test vehicle.
- C. Develop a traffic control plan in accordance with WATCM.
- D. Develop a safety plan.
- E. Establish the safe load thresholds to monitor for during testing.
- F. Outline roles and responsibilities of all personnel involved.

4.0 Load Test

After observing how the structure is expected to behave under the test load and a plan is set on how to monitor the bridge during testing, the actual load test can begin.

- A. Bring all sensors and equipment to site.
- B. Set up traffic control as required for the site.
- C. Install all sensors and testing equipment in accordance with the sensor plan.
- D. Test and calibrate the equipment.
- E. Measure and weigh the axle configuration of the test vehicle.
- F. Mark out test vehicle positions on the bridge deck.
- G. Position test vehicle.
- H. Record structure response. Allow vehicle to remain in place for 30 minutes or until the response has stabilized.
- I. Re-test as necessary to confirm linear behaviour.
 - Should non-linear behavior be observed, the test should be suspended until the cause is determined.
- J. Post process and clean up the data as required after the test is complete.

5.0 Refined Analysis

Similar to the preliminary analysis, the measurements from the load test are used to further refine the FE model and load rating.

- A. Use strain measurements taken during the load test to determine the lateral distribution of live load.
- B. Use measurements taken during the load test to refine the FE model.
 - Adjust various model parameters (i.e. stiffness, support conditions, material properties) until the model produces reasonably similar results to those measured during the load test.
- C. Apply the test load to the refined FE model and extract shear strains at the critical locations.
 - Use the French load spreading method to distribute the shear load across the effective width of the slab.
- D. Calculate refined Live Load Capacity Factor with measured and extracted structure strains.

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