Innovative Overload Permitting in Manitoba
Allowing a 363 250Kg (GVM) “Superload”

Geoffrey C. Oramasionwu, M.Eng., P.Eng., Manitoba Infrastructure and Transportation
Marta E. Flores-Barrios, M.Sc., P.Eng., Manitoba Infrastructure and Transportation
Ryan M. Sanders, B.Eng., E.I.T., Manitoba Infrastructure and Transportation

Paper prepared for presentation

At the “Structures – Transportation: Better – Faster – Safer Session”

2013 Annual Conference of the
Transport Association of Canada
Winnipeg, Manitoba

ABSTRACT

Within the past few years, the demand for hauling heavier loads on highways has increased significantly. Manitoba Infrastructure and Transportation (MIT) has over 1000 Bridge structures on their highway network that are nearing the end of their service life. Faced with an increasing number of overload permit applications and aging structures, a refined method of load rating is needed to permit overloads efficiently while avoiding damage to bridge structures. The Heavy Haul Industry has developed equipment that can control the distribution of truck loads on bridges. MIT uses AASHTO's Non-Standard Gage (NSG) Distribution Factor Method to predict the performance of superstructure elements for bridges. These particular overloads weighing 80% more than the design loads have been reviewed and approved using the NSG method. MIT is the only Agency in Canada that uses the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specification and the corresponding Load and Resistance Factor Rating (LRFR) specification for bridge load rating. AASHTO's Bridgeware rating program (Virtis) employs the NSG method for load rating and using this method, MIT was able to issue permits for safely moving three transformers each weighing 357 000 kg (Gross Vehicle Mass, GVM). Actual GVM's for each load were also confirmed using portable scales. Field monitoring and measurements were undertaken in order to compare the structure's actual performance against theoretical values. This Paper will discuss the details of the load rating analysis using the Non-Standard Gage Distribution Factor Method, the instrumentation used for monitoring, and the results of the comparison between actual and theoretical values. By utilizing Virtis, MIT rated the bridges along a predetermined route and obtained Operating Rating Factors which were greater than one; hence the transformers were successfully moved and upon inspection, the structures behaved as predicted.

INTRODUCTION

Many Bridge Specifications for Design and Load Rating are generally conservative and not necessarily indicative of how bridges will actually perform. MIT uses the AASHTO Standard Specifications for Highway Bridges (AASHTO STANDARD 2007), the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD 2010), and the AASHTO Manual for Bridge Evaluation (AASHTO 2010) which specifies the Load and Resistance Factor Rating (LRFR) methods for design, rehabilitation and load rating of bridges. The goal of this paper is to: (1) present how MIT uses the AASHTO Bridgeware program called Virtis to perform load ratings; (2) present how a "superload" weighing 357 000 kg was evaluated, approved and permitted by MIT; (3) understand how other Canadian jurisdictions analyse a non-standard gage vehicle; (4) demonstrate how Finite Element (FE) or "rigorous" model for calculating distribution factors is a more accurate alternative for reviewing and permitting overloads; (5) present the field measurements and to show how the collected data compares to the theoretically predicted behaviour of beam elements on two of the bridge structures along the permitted route.

OVERLOAD PERMIT REQUEST

MIT received an overload permit application from Equipment Express Inc. to haul three transformers. The three transformers each weighing 175 450 kg were to be hauled from CG Power in Winnipeg, Manitoba to Manitoba Hydro's new Riel Sub-Station. Equipment Express proposed to use two tractors to tow the 175 450 kg transformers on dual lane 12-line hydraulic trailer units. Each axle of the trailer units consisted of 16 tires (8 wheels) and the axles were equally spaced at 4'-11" (1.5m). The initially proposed GVM was 294 823 kg. This configuration distributed the load (including the counterweight on tractors) over a distance of 134 ft (40.884 m). The weight of the transformer was to be equally distributed over all the axles using hydraulic mechanisms.

Fig. 1 – 357 000 Kg Superload on Dual Lane 20-Line PST Hydraulic Trailer with 8 Wheels per Axle

ROUTE INVESTIGATION

The original proposed route (Route No. 1) was 82 kilometres in length and would have crossed seven (7) bridge structures. Because of the width and length of the trucks, proposed speed of travel, and ultimately a safety concern for the travelling public, MIT imposed travel restrictions, allowing them to travel only at night, from 8 pm to 6 am. It would have taken two nights to move these loads along the originally proposed route. This route included roads and structures owned by different jurisdictions. One bridge was a particular concern to the owner; as a result, this route was denied. Because Equipment Express was denied access to that structure by the governing jurisdiction, MIT then selected two alternate routes for review. The first alternate route (Route No. 2) included two timber bridges but the condition of these two bridges warranted further investigation; the preliminary load rating showed that the approach spans would fail in shear. Ultimately, the permit was denied a second time. The second alternate route (Route No. 3) would add an additional night of travel and therefore it would take the carrier a total of three nights to reach their final destination for each move. The structures that were affected along this route are listed below in Table 1.

SITE	STRUCTURE TYPE	ROAD NO	FEATURE_INTERSECT	DESIGN LOADING	DESIGN LOADING GVM (KG)	YEAR BUILT
343	Concrete I-Girder	003	La Salle River	HSS25	59670	2005
327	Concrete I-Girder	059	Red River Floodway	HSS25	59670	2008
2192	Concrete I-Girder	059	Seine River Diversion	HS25	40840	1976
327	Concrete I-Girder	059	Red River Floodway	HSS25	59670	2008
2993	Rigid Concrete Frame	100	CNR Sprague Sub-Way	HS20	32670	1959
3578	Concrete Channel Girder	210	St. Adolphe Coulee	HS25	40840	1997
396	Concrete I-Girder	210	Red River	HS25	40840	1975

Table 1 - List of Primary Bridges

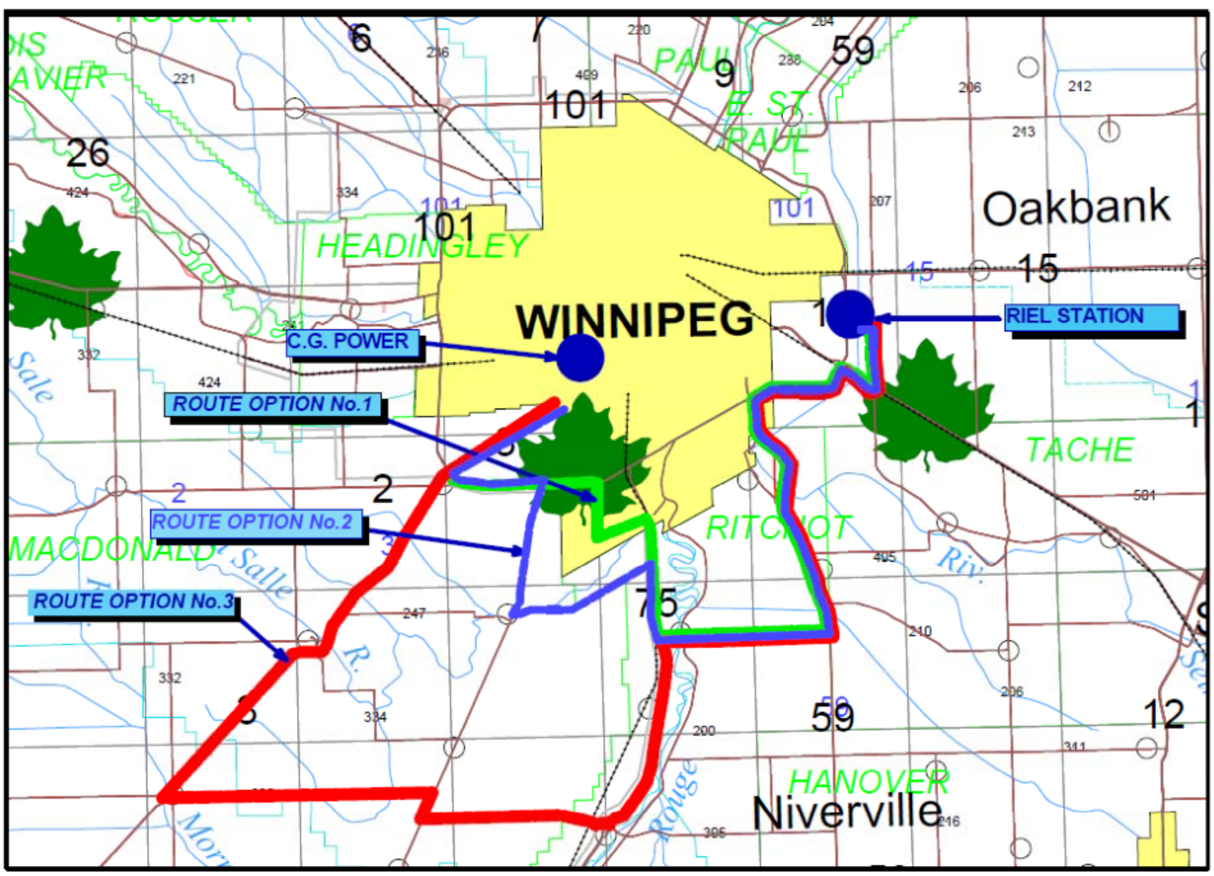


Fig. 2 Map Showing Alternate Routes

It can be seen from table 1 that the GVM of the proposed truck is far greater than the design live loads for each bridge. The Red River Bridge at St. Adolphe shown below consists of seven spans of Precast Prestressed Concrete I-Girders acting in composite action with a 200 mm reinforced concrete deck. Each span has a clear span length of 134 feet (40.845 m).

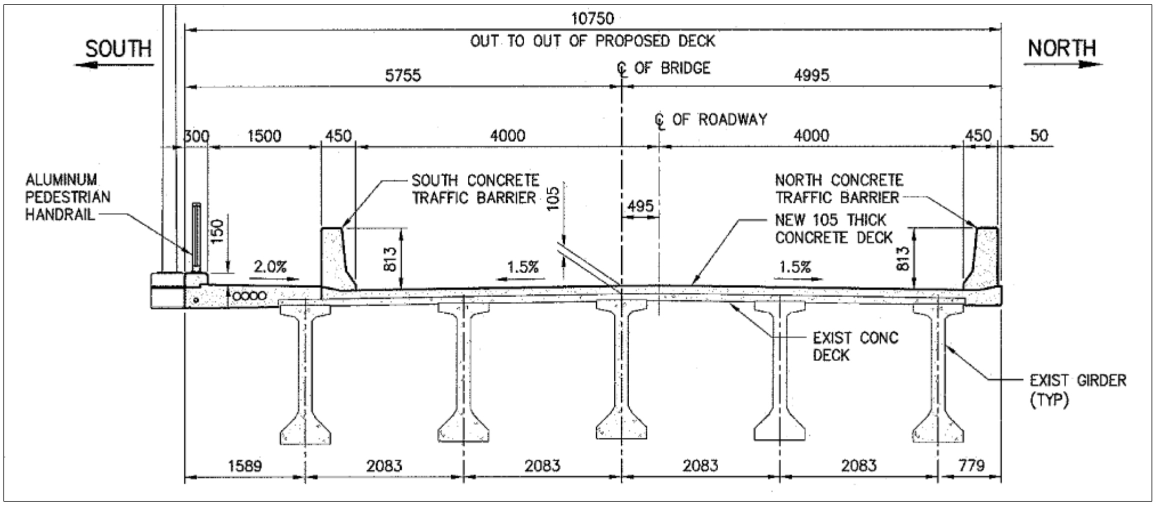


Fig. 3 - Red River I-Girder Bridge Cross-Section

The St. Adolphe Coulee Bridge, shown below, is MIT's standardized 12 m Precast Prestressed Concrete Channel (PPCC) multi-beam bridge with a 100 mm thick asphalt overlay wearing surface and a clear span length of 11.80 m.

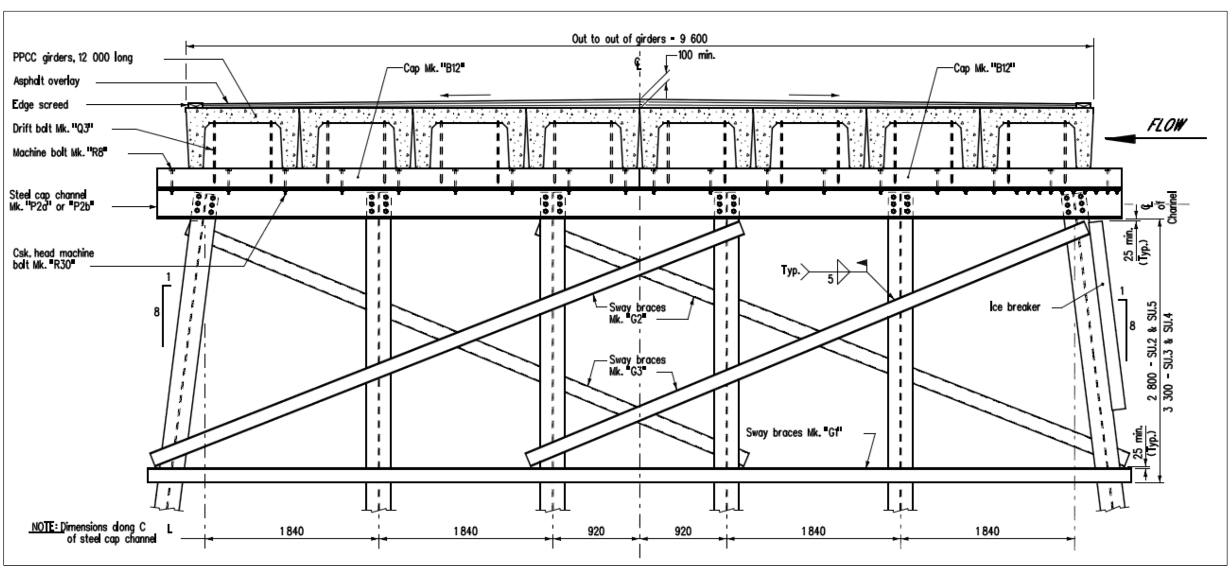


Fig. 4 – St. Adolphe Coulee PPCC Bridge Cross-Section

By observation, the two structures discussed above seemed to have a lower capacity than the rest; therefore these two were chosen for the first round of Load Rating. The truck configuration, shown in Appendix A, was created in the Virtis Bridge Rating database and was treated as a non-standard gage vehicle as defined in the Virtis Training Manual (AASHTOWare 2008). The results of this preliminary load rating showed that the operating rating factors were less than one and that the two bridges would not have enough capacity for the double tractor-trailer configuration. Equipment Express was informed that this configuration would not be permitted on these two bridges. Having exhausted other route options, Equipment Express decided to modify their equipment by increasing the number of trailer units to 18. With the additional trailer units and the two tow tractors, the total truck length became 134 ft (40.845 m) and the proposed GVM became 369 606 kg (Appendix A) It is important to note that the given proposed GVM had not yet been confirmed by MIT. As the investigation and route alternatives progressed, the GVM increased significantly too. Rating the two bridges again with this second proposed trailer configuration yielded failing results yet again on the Red River I-Girder Bridge due to the fact that the entire length of the combined trailer units fit on a single span thereby inducing more force effects on the long span bridge. The St. Adolphe Coulee PPCC Bridge had enough capacity for this configuration because only a maximum of eight of the eighteen trailer units could fit on a single span at a time. This second trailer unit arrangement was also rejected by Water Management and Structures. Equipment Express decided to adjust their configuration again by replacing the two towing tractors with two self propelled power units and also by increasing the number of trailer units to 20; they resubmitted this final arrangement for evaluation. Removal of the two towing tractors with counter weights allowed for the addition of trailer units and therefore enabled the longitudinal distribution of the transformer load. However, adding more trailers added more mass to the GVM but the benefit was being able to distribute the load evenly over a longer length. The final GVM presented to MIT for evaluation was 333 550 kg (Appendix B).

LOAD RATING METHODS

Manitoba Infrastructure and Transportation is the only jurisdiction in Canada that uses the AASHTO LRFD Specification in designing new bridges and Load Rating of existing bridges. The AASHTO Manual for Bridge Evaluation (AASHTO 2010) provides three load rating methods:

1. The Allowable Stress Design Method (ASD) load rating.
2. Load Factor Design (LFD) load rating, and
3. Load and Resistance Factor Rating (LRFR) method, which utilizes the Load and Resistance Factor Design (LRFD) method.

Virtis is used for load rating by Water Management and Structures. The advanced features in this program allowed the rating truck to be positioned on a specific line of travel and calculate the distribution factor for each girder elements of the bridge superstructure. Typically, all AASHTO design trucks, Transport Association of Canada (TAC) legal trucks, Canadian Highway Bridge Design Code (CSA 2006) and most of the overload trucks have a standard or fixed wheel spacing of 6 feet (1800 mm). This is referred to as “Standard Gage” vehicles. The Equipment Express truck described above does not fit the Standard Gage definition for single lane loading. In order to conform to the Standard Gage definition, the truck would need to be considered as two vehicles positioned side-by-side on two separate lanes.

However, the Virtis program offers a different way of handling axles wider than Standard Gage by creating an accurate model of the rating truck based on realistic axle configurations, wheel spacing, and wheel loads. This is referred to as the “Non-Standard Gage” model for Distribution Factors. The main difference in using AASHTO’s non-standard gage method of Load Rating is in the calculation of the distribution factors. After calculating the distribution factors using the non-standard gage model, the load rating analysis is performed similar to the standard rating methods. It is clear that different distribution factors would be obtained for each method. Table 2 on Page 13 compares the differences in distribution factors calculated by each method.

METHODS USED BY OTHER JURISDICTIONS

We wanted to know how other jurisdictions would perform load ratings of their bridges using this overload truck. Seven Infrastructure and Transportation agencies and one Consulting Engineering Firm from across Canada were surveyed to determine their respective process of analysing non-standard gage vehicles for the purpose of load rating. All seven jurisdictions: Alberta, British Columbia, New Brunswick, Nova Scotia, Ontario, Prince Edward Island, Saskatchewan, and the Consultant responded to the survey and provided the requested information. It should be mentioned that all the Provinces involved in this survey follow the CAN/CSA-S6 Canadian Highway Bridge Design Code (CHBDC) for the evaluation and load rating of their bridge structures. Alberta uses a non-standard gage model. Ontario uses a non-standard gage model only if a detailed finite element analysis (FEA) model or grillage is being done. Nova Scotia follows the Ontario Bridge Formula. New Brunswick and British Columbia may require the mover to hire a Consultant for a more detailed analysis for large loads. Saskatchewan uses a non-standard gage model only if a more refined analysis is necessary to permit a specific overload. Prince Edward Island does not consider a non-standard gage model. The Consultant models the non-standard gage vehicle with the actual axle spacing and not according to CHBDC; however they do not consider the number of wheels per axle when load rating, only for design calculations.

CHBDC CODE METHOD

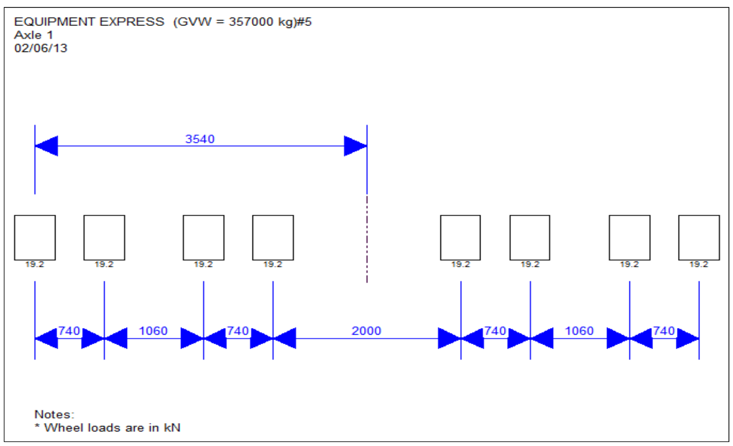
In order to perform a load rating of a non-standard gage vehicle using the CHBDC; dual lane loading could be used but this would still result in more conservative distribution factors and ultimately a less accurate determination of the structure’s reserve capacity.

Section 14.11.4 of the CHBDC refers to the Simplified Method of calculating distribution factors for the purpose of load rating and states: “In this method the lateral distribution is calculated in accordance with the simplified method of Section 5. However, it is possible that the methods specified in Section 5 will not be suitable for non-standard bridges or permit loads (especially those that are wider than the CL-W vehicles), in which cases such methods shall not be used.”

The CHBDC distribution factor method was compared with other simplified methods in the NCHRP Report 592 and was found to be an overly conservative method of computing distribution factors for different bridge structures when compared with the rigorous method. The literature suggests, from the list of structures that needed to be rated, that there is a need for a better method of computing the distribution factors for the proposed live load.

AASHTO BRIDGEWARE METHOD

For bridge structures with reinforced concrete decks in composite action with prestressed concrete I-girders carrying two or more traffic lanes, the AASHTO Standard Specification (1996) distributes the truck live load based on the spacing of the girders. The wheel lines for non-standard gage vehicles can only be described by the number of equivalent lanes in the AASHTO Standard Specification. AASHTO LRFD Bridge Design Specification expressions for live load distribution considers the variations in girder spacing, girder stiffness, span length, vehicle transverse position, skew of the bridge, and slab stiffness.



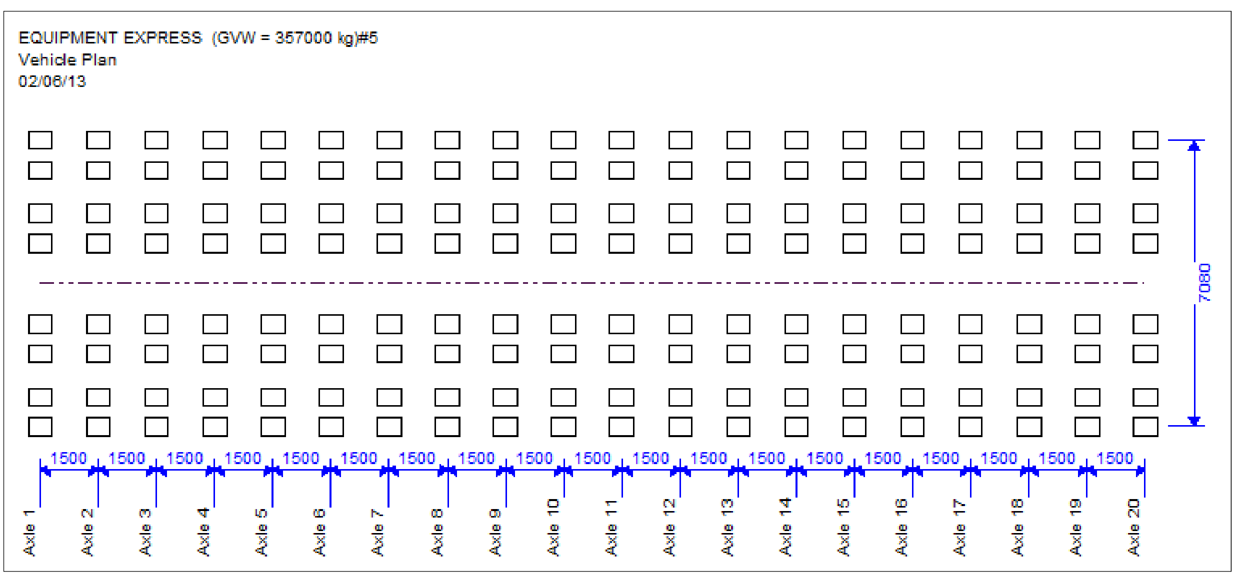


Fig. 5- Graphical outputs for the transverse and longitudinal configuration of the proposed trailer

AASHTO LRFD equations for the distribution factors are more accurate than those provided in the AASHTO Standard Specifications. Paul J. Barr, et al, reported that the LRFD Specification distribution factors can be quite uneconomically conservative for bridges with large span-to-depth ratios. According to AASHTO LRFD Bridge Design Specifications, Section 4.6.2.2.1, "Live load distribution factors, specified herein, may be used for permit and rating vehicles whose overall widths are comparable to the width of the design trucks." In AASHTO LRFD clause 3.6.1.2.2, the specified design trucks have variable trailer axle spacing, but fixed wheel spacing of 1800 mm occupying a design lane of 3600 mm.

The method of distribution suitable for this non-standard gage overload truck has been presented by Paul J. Bar, et al and also in (NCHRP 2007) and is referred to as the "Rigorous Distribution Factor Calculation" (Paul J. Barr 2001). The advanced feature in the Virtis program employs the Rigorous Distribution Factor Calculation Method. In Virtis, the user can create the live load data in the truck library. By using the Graphic User Interface (GUI) in Virtis, the user has the opportunity to position the live load rating trucks along a pre-defined travel path along the bridge deck. For all the bridges rated with these transformer trailers, the only option was to position the trailer's reference line at the centreline of the roadway. In some cases, the width of the trailer was very close to the clear roadway width of the bridge. For instance, the Red River I-Girder Bridge has a clear roadway width of 8m whereas the trailer width was 7.555m (out-to-out of tires), leaving only 0.223m (9 in) on either side of the trailer. In order to calculate the distribution factors, Virtis utilizes both the 3D and 2D finite element models of the superstructure. The deck slab was modeled in 3D as shell elements. Details of the finite element models and subsequent rigorous distribution factor calculations are presented in NCHRP Report 592, Paul J. Barr, et al and Virtis' non-standard gage rating example (AASHTOWare 2008). Graphical outputs for the transverse and longitudinal configuration of the proposed trailer units' database information are as shown in Fig. 5.

The graphical representation shows the wheels are positioned exactly as located on the trailer and the center line represents the point of reference for positioning the trailer on the bridge deck. The line of travel is along the longitudinal length of the bridge and also refers to the trailer's reference line. Virtis allows the user to position the truck to the reference line of travel as opposed to a fixed lane or fixed wheel spacing that is typically used in the design vehicle or legal truck load rating. It is worth mentioning other factors that affect the distribution factors that are calculated with the Virtis Program:

1. **Dynamic Load Allowance (Impact)** – AASHTO Bridge Rating allows the user to alter the value of the dynamic load allowance to reflect the speed at which the vehicle crosses the bridge. Under a closely monitored move where escort personnel and pilot vehicles follow the truck, the speed can confidently be reduced by a certain percentage of the normal posted speed at the service level. In this situation, escorts and pilot vehicles were on hand to monitor the speed of travel across the bridge structures. Hence, it was ensured that the trailers traversed the structures at the required speed of 5 km/h. For a 100 km/h route, the dynamic load allowance factor was reduced to 5/100 = .05 (0.1 factor was entered for this purpose).

2. **Multiple Presence Factor** - Virtis allows users to indicate whether the trailer will be travelling with other lanes loaded or single lane loading. When single lane loading is chosen, again, the value of the distribution factor generally improves when compared with multiple loaded lanes.

When the trailer is provided with escorts and pilot vehicles, the probability of having the adjacent lanes loaded is reduced to zero. Hence, the multiple presence factor does not apply for this application.

3. **Vehicle Scale Factor** – Each axles of the trailer units were weighed prior to the move therefore there is no need to modify these truck axle weights.

Finite Element "Rigorous" Method

The version of Virtis used in this load rating uses STAAD to perform the finite element analysis. The Virtis generated cross-section of the Red River Bridge is shown on Fig. 6. Virtis models the deck slab as shell elements; the element properties have the same physical properties as the concrete deck and the mesh size depends on the desired level of accuracy in the analysis settings, the vehicle longitudinal increment setting, the target length element setting and the target number of shell elements between beam settings. The smaller the size of mesh selected, the slower the computations become and the more accurate the calculated distribution factors become. Each node of the shell element mesh is loaded with a unit load and the finite element engine calculates the "influence surface" for shear, bending moment, deflection, and displacement for that unit load. Hence, the number of load cases depends on the number of shell element nodes that are within the travel lanes.

The 3D models generated by Virtis for the Red River I-Girder Bridge and the St. Adolphe Coulee PPCC Bridge are as shown in Figures 11 to 14. The non-standard gage truck, which was specified to travel along a specific travel path on the bridge deck, is then moved longitudinally at the interval specified in the longitudinal increment setting. Since only one truck is positioned to travel along the specified path, the transverse deck loading is limited to only the locations of the wheels. If any wheel load is positioned adjacent to and not directly over the shell nodes, the load will be transformed to an equivalent nodal point load. At each placement of the wheel load, the corresponding influence surface at that location is multiplied by the wheel load and the summation of all force effects gives the shear, moment, deflection, and displacement envelopes for the beam elements at that longitudinal position. 2D structural elements with nodes at the centroid of the girder and beam elements with material properties of the girders were also generated in Virtis. The beam element model data is also generated for STAAD's structural analysis. A unit point load is also placed on each beam node to generate the influence line force effects as shown in Figures 11 to 14. The full discussion on the calculation of distribution factors using this combination of 3D and 2D elements can be found in (NCHRP 2007).

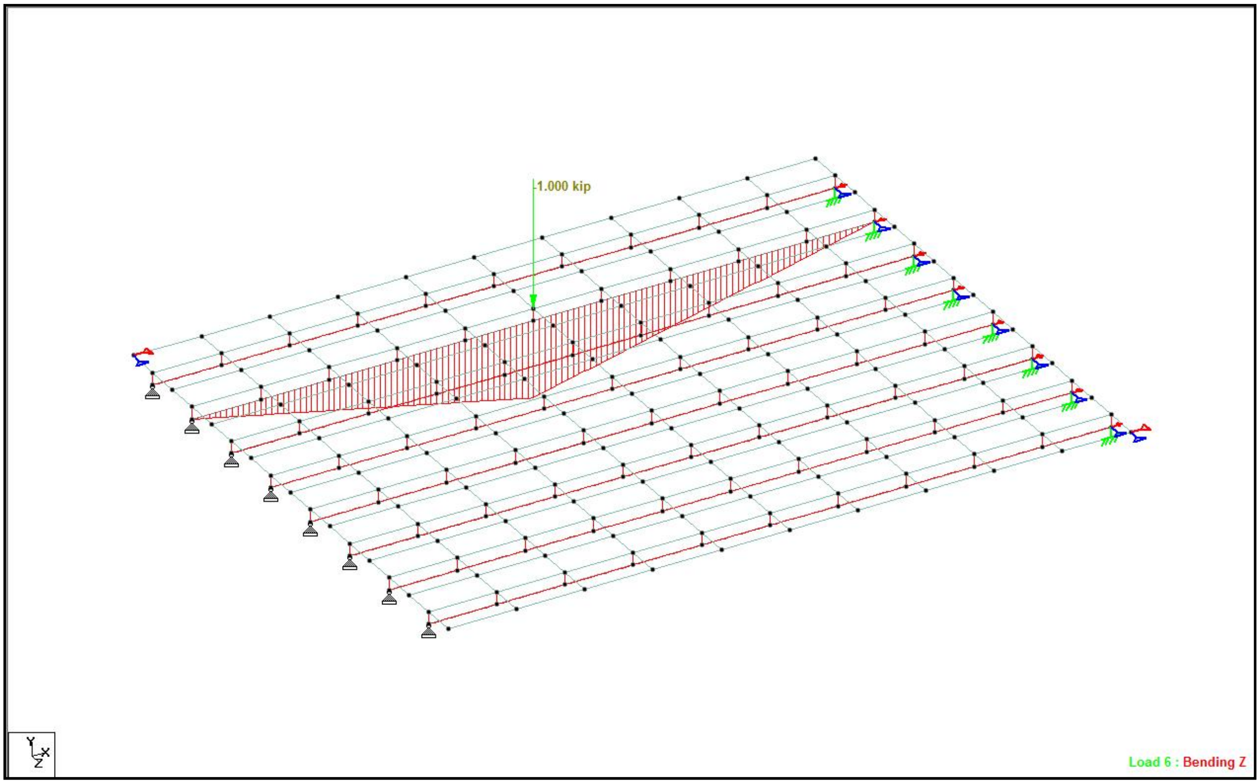


Fig. 7 – Virtis 3D Model of the PPCC Bridge. Note the Bending Moment Envelope for a Unit Load Placed on a Beam Node.

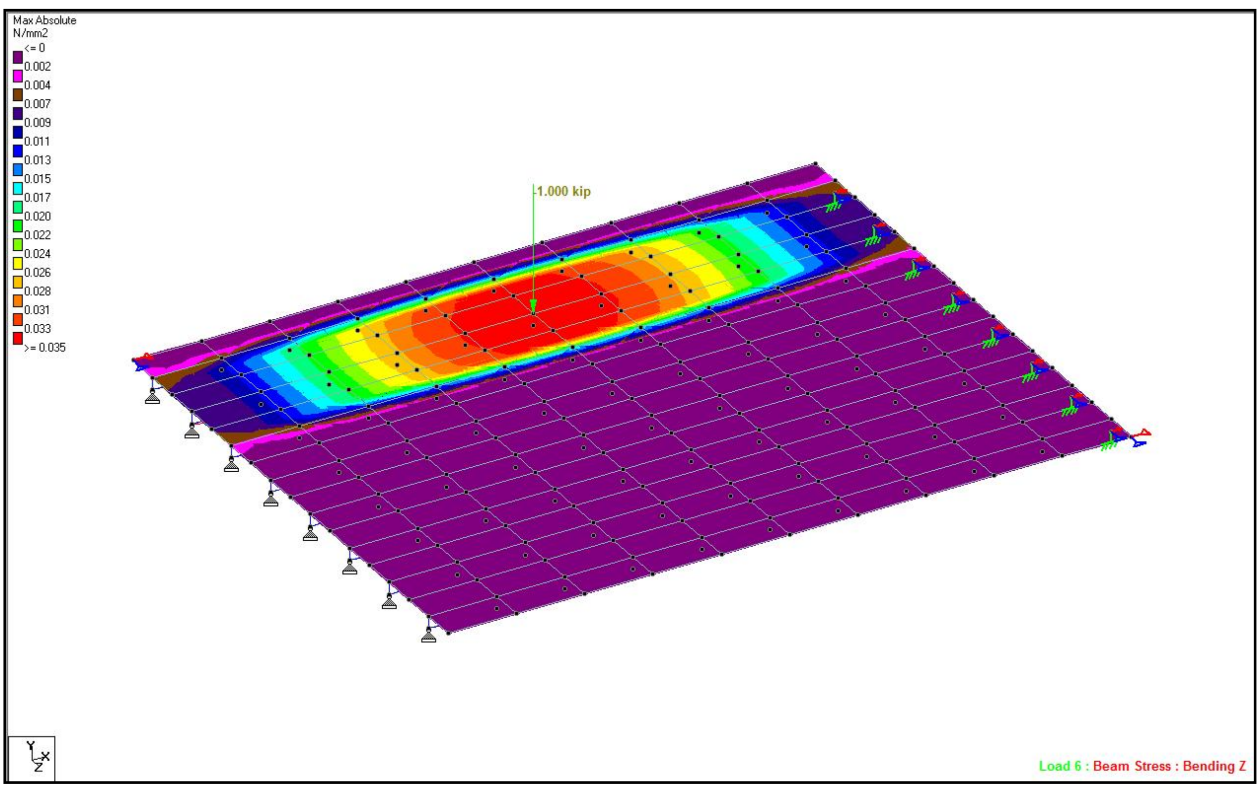


Fig. 8 – PPCC Bridge 3D Stress Influence Surface for the unit Load Case No. 6, Placed at a Shell Node.

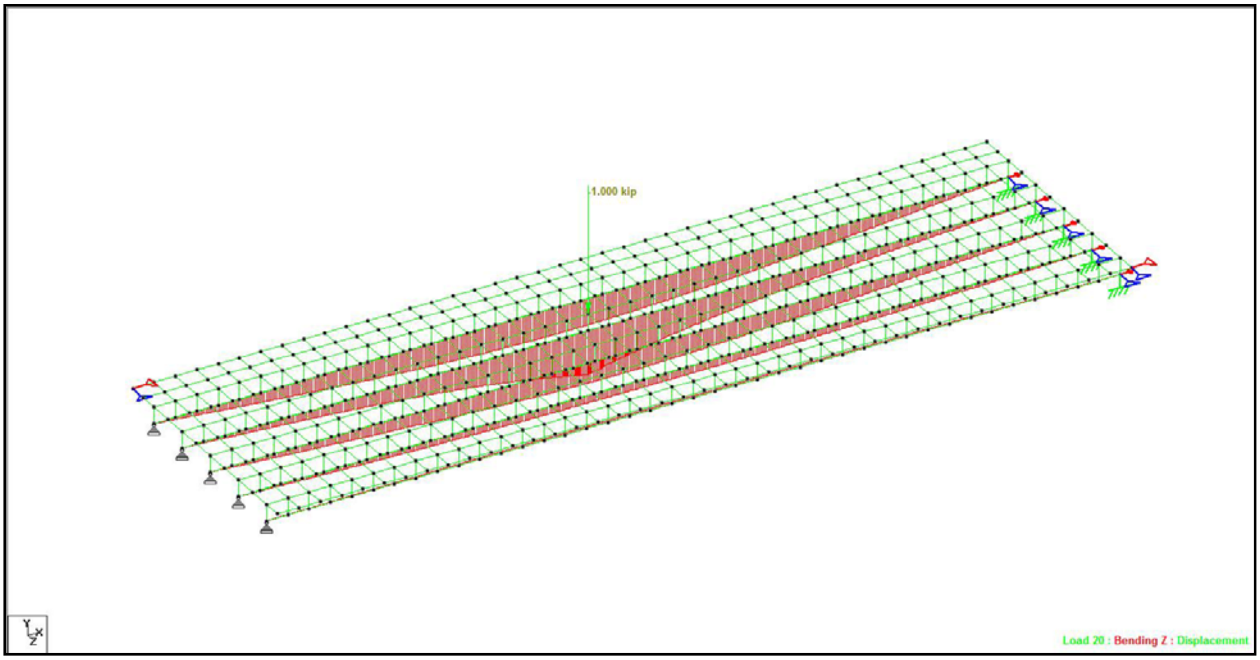


Fig. 9 – Virtis 3D Model of the I-Girder Bridge. Note the Bending Moment Envelope for a Unit Load Placed on a Beam Node.

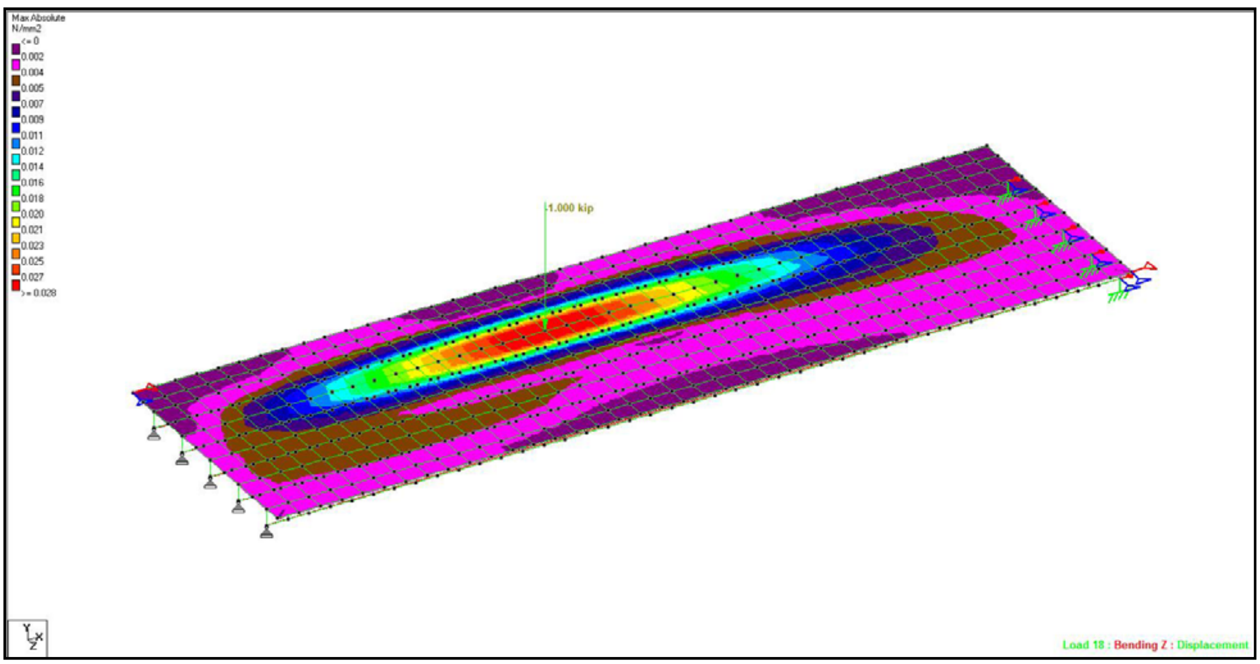


Fig. 10 – I-Girder Bridge 3D Stress Influence Surface for the unit Load Case No. 12, Placed at a Shell Node.

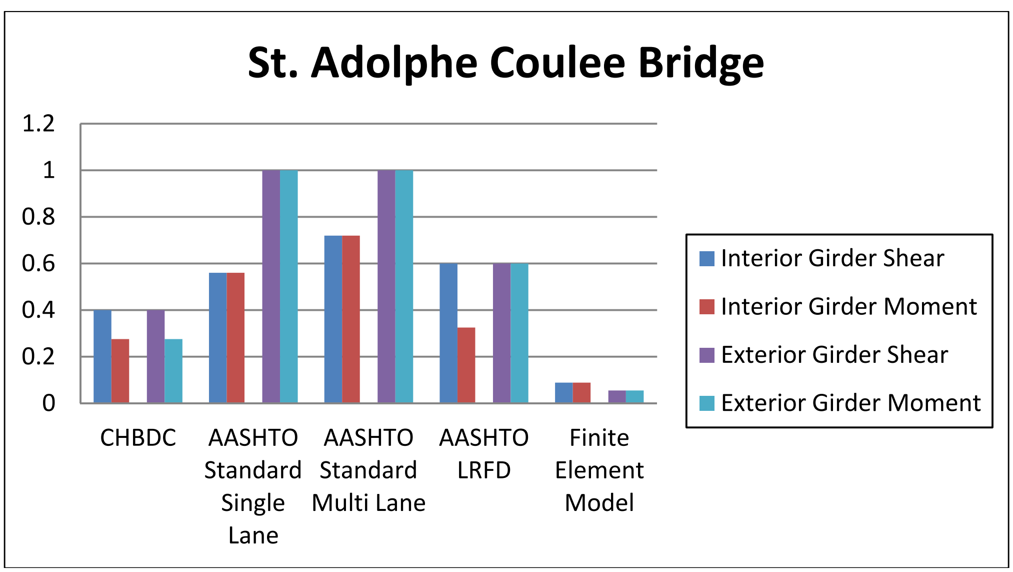
RESULTS

The summary of the distribution factors calculated for each beam on a typical span for the two bridges using this finite element model are given in Table 2. The Table also shows the distribution factors calculated using AASHTO Standard, AASHTO LRFD, and the CHBDC Codes. You will note from the Table that the CHBDC live load distribution factors listed are equal for all the girders (exterior and interior) for the St. Adolphe Coulee PPCC Bridge. The Red River I-Girder Bridge has the same distribution factor for all interior girders and another for both exterior girders. The same distribution factor on all girders is in part due to the CHBDC code equations involving the moment and shear distribution factor calculations. CHBDC Section 5.7.1.2.1.2 and Section 5.7.1.4.1 respectively uses both the average moment and average shear acting across the entire cross section and equally distribute amongst all girders as opposed to moment and shear effects acting on each individual girder line.

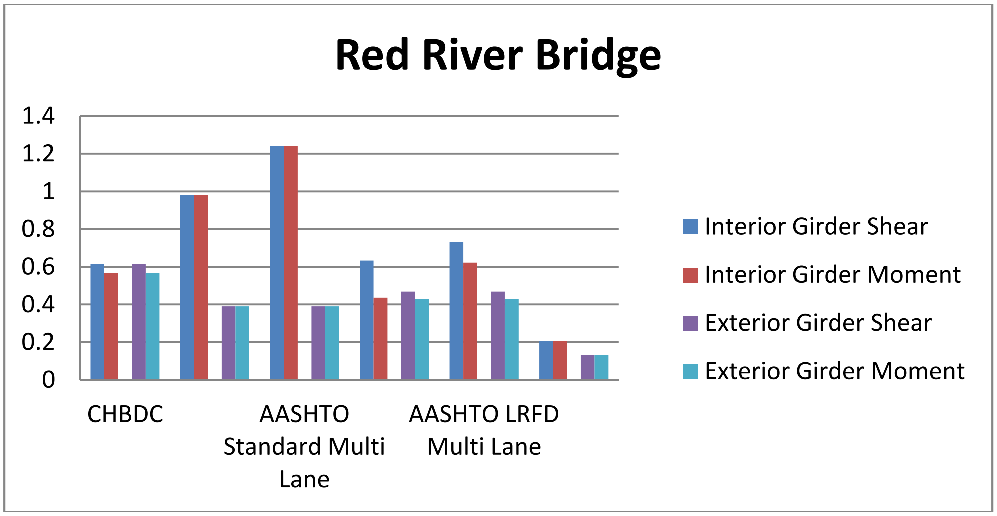
ST. ADOLPHE COULEE BRIDGE					
GIRDER	LIMIT STATE	CHBDC	AASHTO STANDARD Single/two or more Lanes	AASHTO LRFD Single/Multi-Lanes	FINITE ELEMENT MODEL
1	Shear	0.400	1.00/1.00	0.60/0.60	0.055
	Moment	0.276	1.00/1.00	0.60/0.60	0.055
2	Shear	0.400	0.5/0.72	0.60/0.60	0.172
	Moment	0.276	0.56/0.72	0.60/0.61	0.172
3	Shear	0.400	0.56/0.72	0.60/0.60	0.180
	Moment	0.276	0.56/0.72	0.325/0.325	0.180
4	Shear	0.400	0.56/0.72	0.60/0.60	0.089
	Moment	0.276	0.56/0.72	0.325/.0325	0.089
5	Shear	0.400	0.56/0.72	0.60/0.60	0.089
	Moment	0.276	0.56/0.72	0.325/.0325	0.089
6	Shear	0.400	0.56/0.72	0.60/0.60	0.180
	Moment	0.276	0.56/0.72	0.325/.0325	0.180
7	Shear	0.400	0.56/0.72	0.60/0.60	0.172
	Moment	0.276	0.56/0.72	0.325/.0325	0.172
8	Shear	0.400	1.00/1.00	0.60/0.60	0.055
	Moment	0.276	1.00/1.00	0.60/0.60	0.055

RED RIVER BRIDGE					
GIRDER	LIMIT STATE	CHBDC	AASHTO STANDARD Single/two or more Lanes	AASHTO LRFD Single/Multi-Lanes	FINITE ELEMENT MODEL
1	Shear	0.614	0.390/0.390	0.468/0.429	0.131
	Moment	0.567	0.390/0.390	0.468/0.429	0.131
2	Shear	0.614	0.98/1.24	0.633/0.731	0.207
	Moment	0.525	0.98/1.24	0.436/0.622	0.207
3	Shear	0.614	0.98/1.24	0.633/0.731	0.221
	Moment	0.525	0.98/1.24	0.436/0.622	0.221
4	Shear	0.614	0.98/1.24	0.633/0.731	0.231
	Moment	0.525	0.98/1.24	0.436/0.622	0.231
5	Shear	0.614	0.85/0.85	0.582/0.619	0.223
	Moment	0.567	0.85/0.85	0.582/0.619	0.223

Table 2 – Results of Calculated Values of Distribution Factors Using CHBDC, AASHTO LFD, AASHTO LRFD and Finite Element Methods



Graph No. 1 Results of Calculated Values of Distribution Factors for St. Adolphe Coulee Bridge



Graph No. 2 Results of Calculated Values of Distribution Factors for Red River Bridge

Factors Affecting the Accuracy of Finite Element "Rigorous" Method

With F.E. model, the sum of the distribution factors of all girders for a given cross section should add up to unity. As can be seen from Table 2, the sum of the Distribution Factors for the Red River Bridge is 1.013 (1.3 % higher) while that of St. Adolphe Coulee is 0.992 (0.8% lower). The accuracy of F.E. model is affected by the following factors:

- The size of shell elements (or size of the mesh).
- Number of load cases.
- Number of spans and the characteristics of the girder elements.

The higher the number of shell elements (or the smaller the mesh), the more the number of degree of freedom and this increase the accuracy of the analysis but it slows down the processing speed. Hence in order to run these bridges at a reasonable time, the mesh size generator was set to a low level of accuracy.

INSTRUMENTATION AND DATA COLLECTION

The carrier proposed a constant axle weight of 16 678 Kg for the dual lane 20-line hydraulic trailer. One of MIT's concerns was that the carrier would not be able to distribute axle weights evenly as proposed. In order for MIT to verify the actual axle weights, portable scales (see Fig. 11) were used to measure the weight of each axle; the results are shown in Table 12. Figure 7 shows that the actual weight of each axle varied to some extent; the actual GVM were found to be 362 400 kg for the Nov. 03 move, 363 250 kg for the Nov. 14 move, and 357 000 Kg for the Feb. 05/2013 final move. Due to the variability of the GVM of each move it is important to note that MIT revised the load rating for all of the bridges again using these actual weights prior to issuing each overload permit. The St. Adolphe Coulee PPCC Bridge and the Red River I-Girder Bridge were chosen for monitoring during the final move based on the results from the load rating. The Red River structure was solely monitored for deflection using a telescopic measuring pole. As the PPCC structure relies on two bolted connections between each girder to prevent relative vertical displacement at the interface as opposed to lateral post-tensioning with shear keys or a reinforced concrete deck, these girders act more independently than the other structures and therefore don't receive the same load sharing benefit from the adjacent girders. More extensive monitoring was undertaken at the PPCC structure to assist in quantifying the actual live load distribution. Since the transformer move occurred in the middle of a very cold typical Manitoba winter, there were numerous environmental challenges to overcome. Hoarding was constructed for the full width of the structure beneath the mid-span of span 1 with heat provided by a 150,000 btu propane heater. Condensation and run-off from the deck due to heat within the hoarding and high winds combined with -30°C outdoor temperatures and night time conditions were variables that were encountered and overcome. Both the ambient air temperature and the girder concrete temperature were monitored during the installation of the instrumentation and the move. In both cases the ambient air and girder concrete temperatures remained at the low end of the allowable operating temperature of the instruments, +10°C.

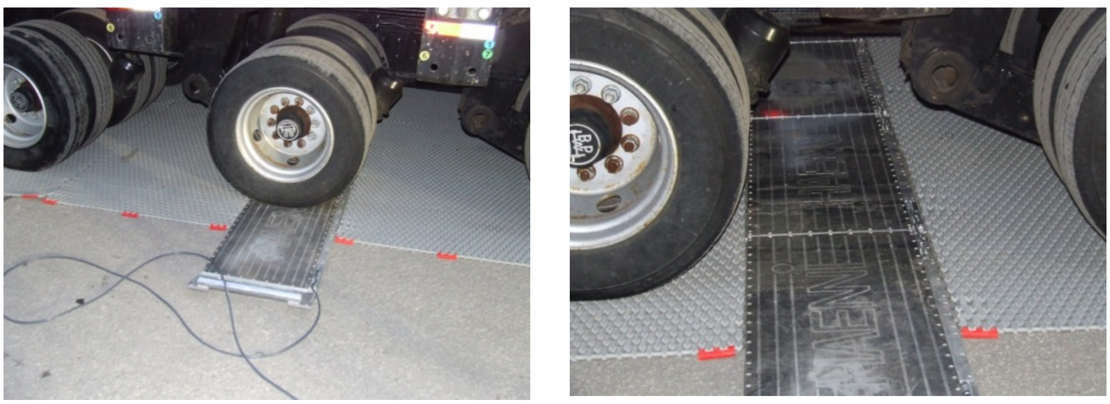
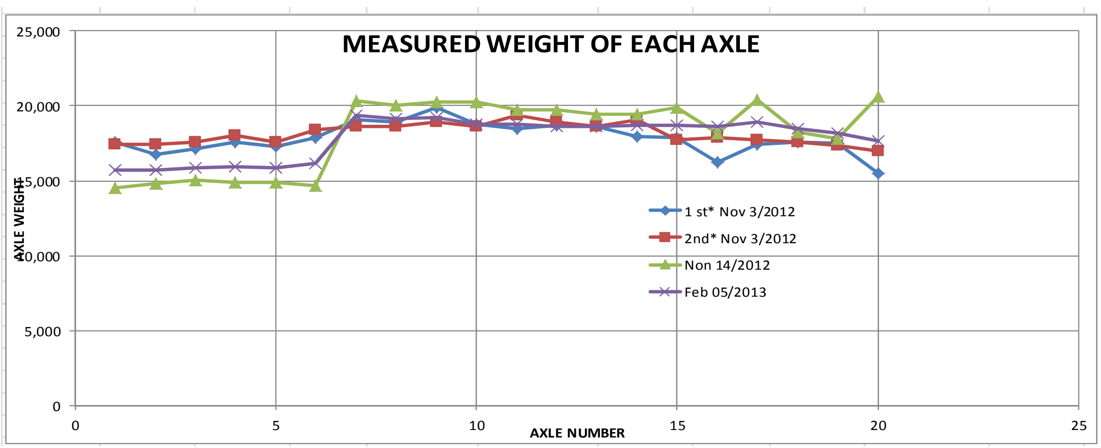


Fig. 11 - Weighing All Trailer Axles Using Portable Scales Prior to Issuing each Permit.



Graph No. 3 - Measured Axle Weights Prior to Issuing Each Overload Permit.

Fig. 13 – Field Instrumentation during the Last Move

A total of four PI-5-100 Displacement Transducers (PI Gauges) were utilized to measure strain at key points along one of the spans. Due to transverse wheel spacing along each axle, girder lines 2 and 7 experienced more of the live load and therefore were chosen as the focus for the instrumentation. Two PI Gauges were installed at the mid-span of each of the two girders, G2 and G7. Each PI Gauge was placed on the underside of each leg of the two channel girders in order to measure the flexural strain imposed by the load. Additionally, two Linear Variable Displacement Transducers (LVDTs) were installed beneath the mid-span of girders G2 and G7, shown on Fig. 14, at the underside of their respective inner legs to measure vertical deflection of the girders as the load traversed the structure. The LVDTs were fixed to tripods and installed completely independent of the structure. A telescopic measuring pole was used to manually verify the deflection in order to ensure the accuracy of the deflection measurements recorded by the LVDTs. Data collection and storage was facilitated by the use of two Model P3 Strain Indicator and Recorder Units complete with two separate laptops.

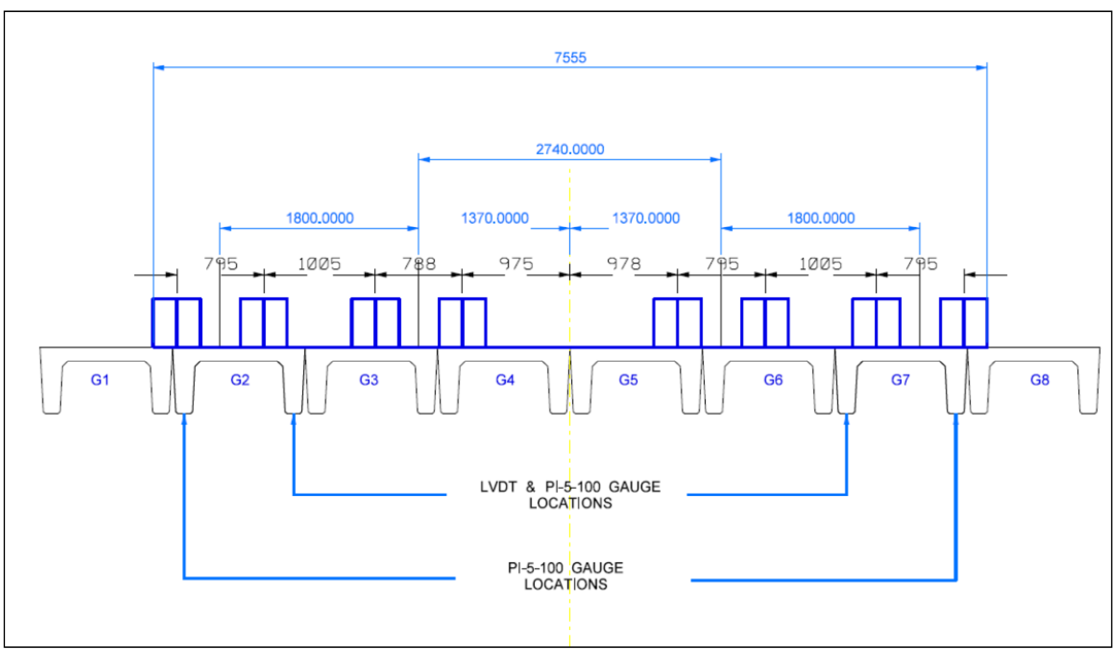


Fig. 14 – Locations of PI and LVT Gauges.

The instrumentation and data collection were redundant in order to compare data and increase the probability of gathering data in the event that there was an equipment malfunction during the move. In theory, as the structure was being loaded symmetrically in the transverse direction, both girders G2 and G7 would experience similar measurements. Therefore each respective side of the structure was outfitted with two PI gauges, one LVDT, one Model P3 Strain Indicator and Recorder, one laptop, and one operator. These redundancies proved to be quite useful as unfortunately during the move, one PI Gauge did not perform as intended and the associated data was unable to be collected.

OBSERVED DEFLECTIONS	G2	G7
Load Rating (Rigorous Method)	19.5mm	19.5mm
LVDT Measurements	5.7mm	6.5mm
Manually Measured	5.8mm	-

Table 3 – St. Adolphe Coulee (PPCC); Comparison of G2 & G7 Measured Deflections with the FEA Theoretical Values.

From Table 3 it is evident that the measured deflections are considerably less than those calculated from the load rating analysis. Although further testing would be required to ensure the accuracy of the measured data, you may note that the values obtained by both the LVDTs and the telescopic measuring pole do coincide.

OBSERVED DEFLECTIONS	G2
Load Rating (Rigorous Method)	40.4mm
Manually Measured	13.5mm

Table 4 – Red River I-Girder Bridge; Comparison of G2 Measured Deflection with the FEA Theoretical Values.

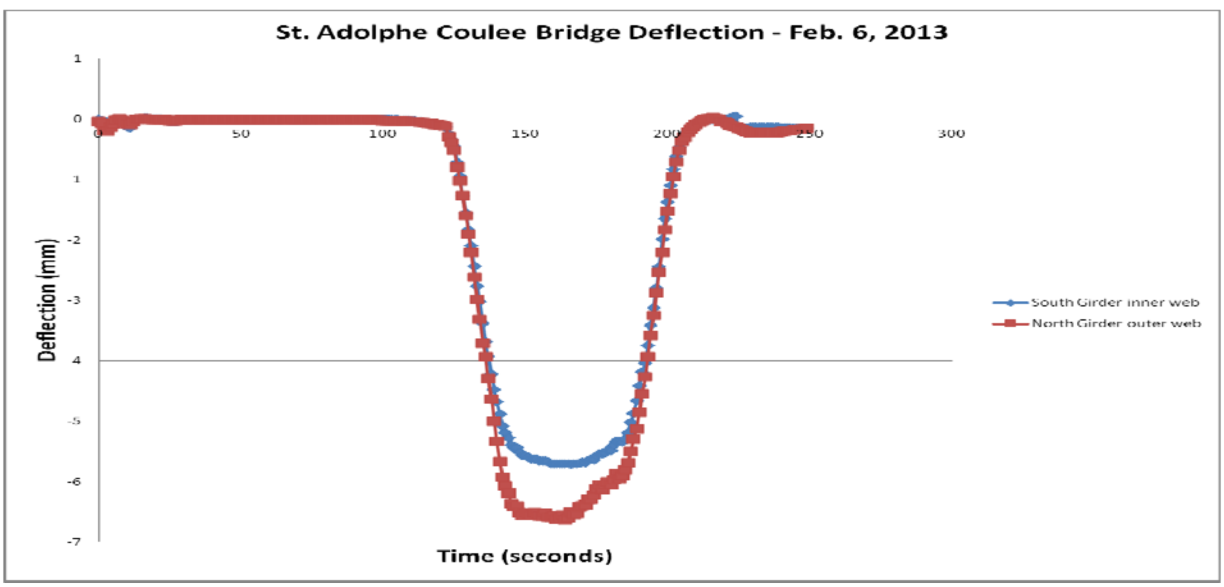


Fig. 15– Differential Deflections on PPCC Girder Webs

For the Red River I-Girder structure, deflection measurements were manually recorded using a telescopic measuring pole. The measured deflection was compared to the theoretical girder deflections that were obtained from the load rating (Table 4); as expected, the actual deflections were lower than the theoretical values. The differences in deflection values listed on Tables 3 and 4 can partially be due to subtle differences in the theoretical load rating model versus the actual structure, such as in-situ material properties versus the design properties utilized for the load rating model. Understandably so, the load rating model also incorporates a certain level of conservatism due to material and load resistance factors to ensure an adequate amount of reserve capacity is available in the structure. Further testing would be required to accurately form conclusive evidence; however in these two instances it is noted that the actual deflections are approximately 30% of the theoretical values determined by the load rating model. Although theoretical and actual strain data was not compared, the measured data from the PPCC structure does indicate differential strains in the legs of each monitored girder. As expected, the strain data suggests that the two monitored girders were rotating inward toward the centerline of the structure along their longitudinal axis. This is consistent with the connectivity of the girders and the applied loads, where the two girders in question were not laterally post-tensioned and were eccentrically loaded relative to their longitudinal centroid.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

MIT was able to permit an overload truck with a gross vehicle mass of 357,000 Kg to traverse seven structures on their highway network; this particular overload was 80% greater than the design load of these structures. This was made possible by employing the Non Standard Gage Method to determine the distribution factors and ultimately completing the load rating using AASHTO's Bridgeware rating program, Virtis.

The following summarizes the key conclusions based on our assessment:

- F.E. (Rigorous) method of calculating the distribution factors of bridges for load rating offers a realistic alternative for predicting the reserved capacity of a given bridge superstructure for and given live loads.
- Live Load configurations outside of the Standard Gage setting can be easily rated with the aid of Finite Element distribution calculation method.
- In-situ material properties of the rated superstructures were not used in the aforementioned load ratings; however, it was observed that in-situ material properties, the presence of curbs, sidewalks, barriers, and railing will increase the stiffness of the bridge superstructure. This is reflected in the disparities between the theoretical and actual deflections and shows that the Rigorous Method of calculating distribution factors is still conservative as there is excess reserve capacity remaining in the superstructure.
- Both CHBDC and AASHTO are very conservative compared to the Finite Element Method.
- Use of instrumentation on structures is beneficial from both overload permitting and general structure performance perspectives.
- Alternative routes are beneficial when attempting a move with many constraints.

- Good communication is essential among governing jurisdictions, clients, and heavy haulers.

Recommendations

- When non-standard gage trucks are utilized for moves due to the size of the overload, the Rigorous Method using Finite Element Modelling should be used to accurately determine the effects of the proposed truck configuration and load on the structures. In certain instances, the Rigorous Method may also be warranted for standard gage trucks.

- Since the magnitude of this particular overload is the first ever for MIT and has enabled us to gain valuable experience in permitting loads of this magnitude, we hope that other agencies can benefit from the lessons we've learned and the information presented in this paper.

REFERENCES

AASHTO LRFD, The American Association of State and Highway Transportation Officials. *AASHTO LRFD Bridge Design Specifications.* The American Association of State Highway and Transportation Officials, 2010.

AASHTO STANDARD, American Association of State and Transportation Officials. *AASHTO Standard Specifications for Highway Bridges, 16th Edition.* American Association of State and Transportation Officials, 1996.

AASHTO, American Association of State Highway and Transportation. *The Manual for Bridge Evaluation 2nd Edition (MBE).* American Association of State Highway and Transportation, 2010.

AASHTOWare, Technical Support Support. "DF1- Distribution Factor Analysis (NSG) Example." *VIRTIS TRAINING*, 2008: 1-16.

CSA. *CAN/CSA-S6, Canadian Highway Bridge Design Code.* CSA, 2006.

NCHRP Report 592, NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM. *Simplified Live Load Distribution Factor Equations.* Washington D.C.: TRANSPORTATION RESEARCH BOARD OF THE NATIONAL ACADEMIES, 2007.

Paul J. Barr, Marc O. Eberhard, and John F. Stantom. "LIVE-LOAD DISTRIBUTION FACTORS IN PRESTRESSED CONCRETE GIRDER BRIDGES." *JOURNAL OF BRIDGE ENGINEERING*, 2001: 1-9.