Design and Performance of GFRP Reinforced Bridge Decks in NOVA SCOTIA—Preliminary Analysis

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1 Introduction

The durability of bridge decks largely influences the lifespan of bridge structures. Environmental factors such as high pH, salt water, high temperature, freeze–thaw cycles and wet/dry cycles cause long-term durability issues in concrete bridge decks leading to a reduced lifespan (Kim et al. 2012). Advanced composite materials such as fibre-reinforced polymers (FRP) have been used as reinforcing material for bridge decks to tackle the effects of adverse environmental conditions (Benmokrane et al. 2020). Glass fibre-reinforced polymers (GFRP) bars are the most frequently used type of FRP bars in bridge engineering mostly because they are high strength, lightweight, non-corrosive and economical, making it ideal for use in bridge environments (ISIS 2007).

The Canadian Highway Bridge Design Code, CSA S6-19, permits the use of FRP bars, and practicing engineers have been designing with FRP as the primary concrete deck reinforcing material for the last two decades. Despite the significant benefits of FRP bars, there is some uncertainty concerning the long-term performance of the material which resulted in having most codes include an 'environmental factor' in the calculation of the capacity of FRP-reinforced members. The durability of FRP bars used in concrete decks still needs to rely heavily on lab testing and statistical

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B. Benmokrane et al. (eds.), *8th International Conference on Advanced Composite Materials in Bridges and Structures*, Lecture Notes in Civil Engineering 267,
https://doi.org/10.1007/978-3-031-09409-5_7

analysis for any quantitative data on performance, but this can be supplemented by continual inspection and assessment of existing structures.

The Center for Innovation in Infrastructure (CII) at Dalhousie University is currently conducting a research programme in collaboration with Nova Scotia Department of Transportation and Infrastructure Renewal (NSTIR) to propose guidelines for designing durable bridge decks in Nova Scotia (NS). The research consists of two interrelated phases. Phase I consists of categorizing and evaluating FRP-reinforced bridge deck design practice in NS over the past 20 years, while Phase II consists of developing a framework to assess the structural reliability of FRP-reinforced bridge deck design options subjected to the province's specific environmental exposure. In this paper, parts of Phase I and Phase II are presented.

2 Phase I: Analysis of GFRP-Reinforced Bridge Decks in Nova Scotia (NS)

A database comprising of the design details of select bridges in NS was developed, summarizing the bridges into categories such as date of construction, abutment type, girder type, concrete compressive strength, deck thickness, span length and other relevant categories. Information for these bridges was obtained, such as stamped engineering design drawings, inspection reports and strength testing reports. This database currently consists of 20 bridges with parameters summarized in Table 1.

Five (5) bridges with FRP-reinforced bridge decks were selected and analysed for the purpose of this paper based on geographical location to capture different regions in NS. The five bridges were selected from the Northern, Central, Cape Breton and South Shore regions of the province. Table 1 also shows a summary of the parameters of the five bridges selected for analysis.

2.1 Analysis Basis

The data collected from the five bridges selected were analysed to determine the demand, capacity and utilization ratios (U.R.) for various parts of the bridge deck. The analysis basis used for the bridges is described as follows:

- All analyses and design checks were performed in accordance with the Canadian Highway Bridge Design Code (CSA S6-19).
- Project details and designs were extracted from approved and stamped engineering design drawings.
- Design loads were taken from methods specified in Sect. 3 (Loads) of CSA S6-19.
- The Flexural Method of evaluating bridge decks as described in Sect. 5 (Methods of analysis) of CSA S6-19 was used to evaluate the flexural capacity of the bridge decks.

Table 1 Summary of bridges in database

Parameter	Value/range	Number of bridges	
		Database (20)	Analysed for this paper (5)
Date of construction	2011–2015	12	2
	2016–2020	8	3
Abutment type	Integral abutment	18	5
	Semi-integral abutment	2	0
Girder type	New England Bulb Tee (NEBT)	15	5
	Box girder	3	0
	Next beam type B	1	0
	28F Next beam	1	0
Concrete compressive strength	45 MPa	19	5
	50 MPa	1	0
Deck thickness	175 mm	1	0
	200 mm	1	0
	225 mm	14	4
	250 mm	4	1
Span length	15–24 m	3	0
	25–34 m	5	2
	35–44 m	9	3
	45–54 m	0	0
	55–65 m	3	0

- Methods pertaining to the design of FRP-reinforced bridge decks were used in accordance with Sect. 16 (Fibre-reinforced structures) of CSA S6-19.

2.2 Analysis Results and Discussion

The summarized results of the analyses are presented in Table 2, showing the utilization ratios (U.R.) for both positive and negative moments in the bridge deck's interior and exterior spans. The concept of a utilization ratio was also adopted in evaluating crack width calculations. The utilization ratios were calculated by dividing the load effect (factored bending moment load or crack width) by the applicable resistance (factored bending moment resistance or acceptable crack width limit).

Table 2 Utilization Ratios (U.R.) for the five bridges analysed

Span	Loading direction	Utilization ratio for bridges					Mean	Standard deviation
		1	2	3	4	5		
Interior spans	Negative transverse bending	0.80	0.51	0.42	0.78	0.63	0.63	0.17
	Positive transverse bending	0.51	0.31	0.38	0.56	0.51	0.45	0.10
	Positive longitudinal bending	0.53	0.35	0.29	0.60	0.56	0.47	0.14
Exterior spans	Negative transverse bending	0.79	0.66	0.46	0.53	0.59	0.61	0.13
	Negative transverse bending—barrier load	0.77	0.70	0.55	0.67	0.63	0.66	0.08
Interior spans	Crack width: negative transverse	1.54	0.99	0.88	1.34	0.88	1.13	0.30
	Crack width: positive transverse	0.50	0.22	0.68	0.60	0.55	0.51	0.18
	Crack width: positive longitudinal	1.01	0.71	0.63	1.20	1.09	0.93	0.25
Exterior spans	Crack width: negative transverse	0.81	0.70	0.51	0.45	0.46	0.59	0.16

As seen in Table 2, the critical zone for bending in the interior spans of the bridge deck on average is the negative transverse direction, with a mean U.R. of 0.63. This corresponds to the location of the largest bending moment in the interior spans in the negative transverse direction. For exterior or cantilever spans, the negative transverse moment caused by barrier loads has the highest mean U.R. of 0.66. The U.R. for crack widths in parts of the deck undergoing negative bending in the interior spans have a mean value of 1.13 which is above 1.0, signifying that on average, the theoretical crack widths are greater than the specified limit, 0.7 mm, as specified in the CSA S6-19 flexural design method. It should be noted that the actual crack widths in these zones have not been verified in the existing structures due to the presence of asphalt wearing surfaces.

These findings indicate that the portions of bridge deck over the interior girders subjected to negative bending are the critical region for the designer to ensure that the transverse crack width criteria in the code is satisfied in the flexural design approach. The designer's choice of the flexural design method versus the empirical design method will result in significant differences in the amount of reinforcement selected at the identified critical region. Also, in the empirical method, the need for a crack width check is waived. The choice of the design method is not available on the drawings so it could not be verified for this study. However, Khanna et al. (2000) and Mufti et al. (1999) have demonstrated, both experimentally and theoretically, that the top layer of reinforcement is not critical to the strength and safety of the bridge deck under wheel loads.

3 Phase II: Reliability-Based Model

A framework was established to develop a reliability-based model that will be used to propose a set of durable bridge deck design criteria for NS. This framework involves performing Monte-Carlo simulations based on identifying and categorizing the factors that contribute to the factored moment demand, M_f, and the factored moment of resistance of the concrete bridge deck, M_r, at ultimate limit state, and identifying the corresponding statistical parameters (mean, bias, standard deviation). The reliability-based model is currently in the early stages of development. Preliminary discussion about the live loads used in the model is provided in this paper.

The maximum wheel load from the live load, P, shall be compared to the maximum wheel load as specified in CSA S6-19 to establish a bias ratio (i.e. actual maximum wheel load divided by the code specified maximum wheel load), determine the distribution type and quantify the variance of the live load. Real-life live load data was needed to conduct a live load analysis of trucks in NS.

A year's worth of weigh-in-motion (WIM) data was received from the NSTIR at a truck scale site in Nova Scotia. This data consists of Class 13 (7 or more axles), Class 12 (6-axle) and Class 11 (5-axle) vehicles, separated into the number of axles, axle weights and distance between axles. With Class 13 vehicles being the highest weight class, more emphasis was made on its categorization and analysis. With approximately 33,300 Class 13 vehicles received, a histogram was created to capture the distribution of the data. This distribution will be used to predict the maximum wheel load for 1-in-75 and 1-in-2 return period.

Figure 1 shows the histogram of the total weights, in kilonewtons (kN), of Class 13 vehicles. It can be seen in this figure that three peaks are present, with two being more visibly prominent. It was recognized that the three peaks show the weight distribution of unloaded trucks, partially loaded trucks and fully loaded trucks (Schmidt et al. 2016). The peaks for unloaded and fully loaded trucks are very distinct and can easily be distinguished, whereas the peak for partially loaded trucks is short and has a wider variance. This is because a significant number of trucks on the road are neither completely empty nor completely full, and a lot of trucks fall somewhere in-between as the data suggests.

After further investigation, the axle loads were categorized into histograms and the same three-peak trend was found for most of the axles. Axle 1 showed a single peak which meant that the load in the back of the Class 13 trucks does not affect the loads in the first axle. A normal distribution probability density function (PDF) was fitted to the data from Axle 1.

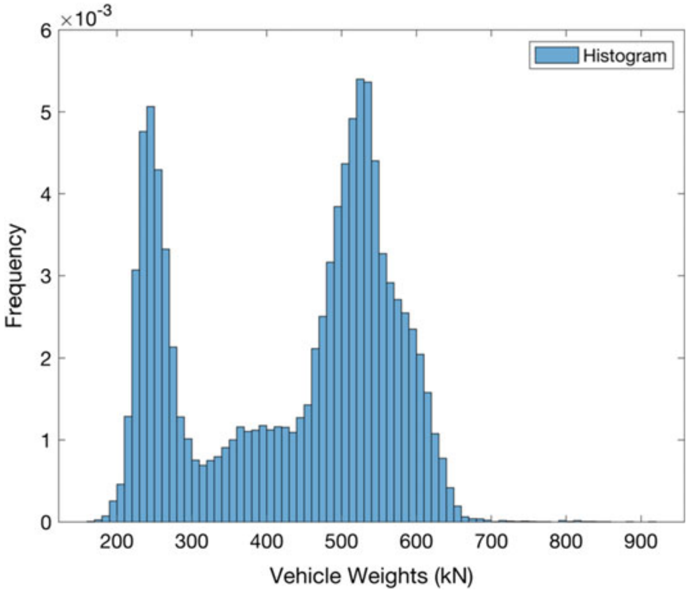

Fig. 1 Histogram of total weights of Class 13 vehicles

The other axles (2–9) showed the three-peak behaviour as well and three normal distributions were fitted to each peak to get a mean value and standard deviation for each. Axle 3 was found to have the largest mean value for the fully loaded trucks (third peak), with a mean of 89 kN and a standard deviation of 5.69 kN. The third peak of Axle 3 is of interest because it shows the maximum axle load of fully loaded trucks and therefore will be used to establish the bias with the maximum value specified in CSA S6-19 once the values have been extrapolated for larger return periods. The probability density function (PDF) and histogram for Axle 3 are shown in Fig. 2.

4 Conclusion

Bridge structures are often under adverse environmental conditions that could reduce their lifespan, and the use of GFRP-reinforced bars helps tackle some of the durability issues that exist. The long-term objectives of this research programme are to provide a critical review of the design methods for GFRP-reinforced bridge decks in Nova Scotia and recommend reliability-based regional specific durability-based design criteria for GFRP-reinforced bridge decks. This paper presents parts of the two-phase approach taken by the research group.

In Phase I, a database was created to summarize the properties and characteristics bridges in NS, where five bridge decks were analysed to obtain the utilization ratios (ratio of demand-to-capacity) at critical locations within the bridge deck. The

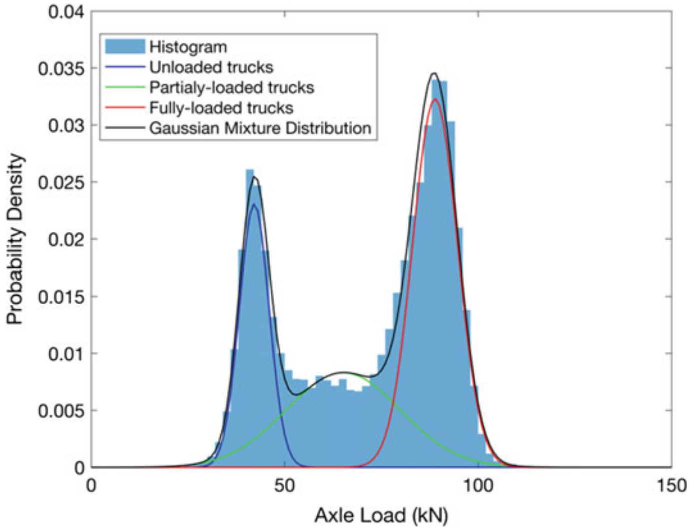

Fig. 2 PDFs and histogram of Axle 3 loads of Class 13 vehicles

utilization ratios for bending moments were below 1.0, ranging from 0.29 to 0.80. The max average U.R. for bending moments was found in the negative transverse loading direction caused by barrier loads, having a value of 0.66. While for crack widths, the max average U.R. was found in the negative transverse bending direction, with a value of 1.13, which is greater than 1.0 signifying that this zone could be the area of the deck most susceptible to cracking.

In Phase II, the live load data used in developing the reliability-based model was briefly discussed. The live load data from a truck scale in NS was obtained and normal distributions were fitted to the axle loads. Most of the axles displayed a three-peak trend which signified that some of the trucks were either unloaded, partially loaded, or fully loaded. Axle 3 of Class 13 vehicles were found to have the highest average weight of all the fully loaded trucks and will be extrapolated for larger return periods when used in the reliability model.

5 Future Work

The statistical parameters for other variables identified in the reliability-based framework will be obtained either from research literature, lab testing, or analytical analyses to perform the Monte-Carlo simulations required for the model. More research will be done on the live load data to extrapolate the maximum axle load for longer return periods: 2-years for evaluating existing bridges and 75-years for designing new bridges. The research team plans to perform lab testing on concrete beams reinforced with GFRP bars that have been subjected to the local Nova Scotia environment and weather conditions for the last 12 years to account for any deterioration or degradation in the GFRP bars and concrete. This combined with previous research literature will help tackle the durability issues faced by concrete bridge decks in Nova Scotia.

Acknowledgements The research team would like to thank the Nova Scotia Department of Transportation and Infrastructure Renewal for approving and sponsoring this project, alongside Dalhousie University.

References

Benmokrane B, Brown VL, Ali AH, Mohamed K, Shield C (2020) Reconsideration of the environmental reduction factor CE for GFRP reinforcing bars in concrete structures. J Compos Constr 24(4):06020001

CSA S6-19, The Canadian highway bridge design code. Canadian Standards Association, Toronto (Ontario)

ISIS, Design Manual No. 3 (2007) Reinforcing concrete structures with fibre reinforced polymers. Intell Sens Innov Struct Can 449–458

Khanna OS, Mufti AA, Bakht B (2000) Experimental investigation of the role of reinforcement in the strength of concrete deck slabs. Can J Civ Eng 27(3):475–480

Kim YH, Trejo D, Gardoni P (2012) Time-variant reliability analysis and flexural design of GFRP-reinforced bridge decks. J Compos Constr 16(4):359–370

Mufti AA, Newhook JP, Khanna OS (1999) Predicting the punching behaviour of reinforced concrete bridge decks. In: Proceedings of the Canadian society for civil engineering annual conference, Regina, vol I, pp 325–334

Schmidt F, Jacob B, Domprobst F (2016).Investigation of truck weights and dimensions using WIM data. Transp Res Procedia 14:811–819