Title: A Review of the Fatigue Limit for Steel Bridges under Ontario Highway Traffic Loading

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Abstract

In many design codes for roadway bridges, it is required that a design truck be passed over influence lines for stress at various locations on the bridge to obtain nominal stress ranges for design. For the fatigue design of Canadian bridges in the infinite or very long-life domain, the nominal stress range is compared with a fatigue limit, after modification by an appropriate correction factor to account for the difference between the nominal stress range and some measure of the extreme stress range in the expected real traffic histogram, which governs infinite life design. The extent to which the influence of simultaneous truck crossings was considered in the establishment of the correction factors is believed to be limited. With this in mind, a simulation-based study, conducted to investigate the effects of simultaneous vehicle crossings on the fatigue limits for steel bridges under Ontario highway traffic loading, is presented in this paper.

Keywords: Fatigue limit, Infinite life, Steel bridges, Fatigue design, Truck traffic

Introduction

In many design codes for roadway bridges, it is required that a design truck be passed over influence lines for stress at various locations on the bridge to obtain nominal stress ranges for design (AASHTO 2017; CSA 2019). For the fatigue design of steel bridges in Canada, fatigue correction or "damage equivalence" factors are employed to relate the damage due to the design or code truck to the damage due to the real traffic loading. Regular calibration of these factors to account for possible changes in the weight of the trucks in the real traffic is crucial. Apart from the possible changes in the weight of the trucks, changes in traffic flow rates can also affect the rate of fatigue damage for steel vehicular bridges. With higher truck traffic flow rates, the spacing between the trucks decreases, and the possibility of simultaneous truck crossings increases. Stress ranges caused by simultaneous truck crossings (i.e., crossing events involving multiple trucks on the bridge at the same time) can be considerably higher than stress ranges caused by single truck passages. This effect is more significant for elements with long influence lines or elements that carry the traffic loads from multiple lanes. Figure 1 shows examples of truck traffic on Highway 401, Ontario. Short spacings can be seen between the trucks in each lane; also, multiple trucks are moving side by side in adjacent lanes.

Figure 1. Truck traffic on Highway 401, Ontario, October 2020.

Typically, two separate fatigue correction factors are defined in bridge design codes - one for finite life and another for infinite life design. (Hirt et al. 2006) presented a simulation-based approach to obtain the correction factors for finite life design. These factors can be determined employing the influence line for a given location along the bridge span, a real traffic database, and a code truck model. (Coughlin and Walbridge 2011) calibrated the finite life factors in North American bridge design codes for welded aluminium highway structures and discussed the effects of rare, exceptionally heavy trucks on these factors. (Walbridge et al. 2013) investigated the effect of simultaneous truck crossings on the finite life correction factors, as well as the effect of the S-N curve slope (m) on this factor, which – while held constant for steel fatigue design at $m = 3.0$, varies depending on the detail category for the fatigue design of aluminum bridge structures. (Chehrazi et al. 2022) obtained fatigue correction factors for bridge elements with very short influence lines, such as expansion joints or orthotropic steel deck panels.

For fatigue design in the infinite or very long-life domain, the nominal stress range is compared with a fatigue limit, after modification by an appropriate factor to account for the difference between the nominal stress range and some measure of the extreme stress range in the expected real traffic histogram, which governs infinite life design. This correction factor is 1.75 in the US currently (AASHTO 2017) and 0.52 / 0.5 = 1.04 for normal bridge elements and 0.62 / 0.5 = 1.24 for bridge decks and expansion joints (where 0.52 and 0.62 are the fatigue correction factors for finite life design) according to the Canadian bridge design code (CSA 2019 (Clause 10.17)). The differences between these numbers stemming largely from differences in the code truck model used to establish the nominal stress range for fatigue design. The extent to which the effect of simultaneous vehicle crossing was considered in the establishment of the correction factor for infinite life design in the North American context is believed to be limited. Furthermore, it has been hypothesized that these effects may be even more significant in the infinite life domain, where relatively rare load events can dominate fatigue behaviour. With traffic volumes increasing, the infinite life domain is increasingly being the more relevant scenario for roadway bridges. In Ontario, Canada, for example, all vehicular bridges are now being designed for fatigue assuming the highest traffic volume category by default (Highway Class A, 4000 trucks/day), in anticipation of future growth. With this in mind, a simulation-based study, conducted to investigate the effects of simultaneous vehicle crossings on infinite life correction factor for steel bridges under Ontario highway traffic loading is presented in this paper. Furthermore, the effect of changes in the weight of trucks in the real traffic database is discussed and the infinite life factors are calculated for a broad range of influence line lengths (i.e. bridge spans).

In the following sections of this paper, the approach used to simulate the real, in-service cyclic stress histories in bridges is first described. This approach has been implemented in a FORTRAN code, which has been adapted for the current study to simulate real traffic flow, including simultaneous truck crossing events. Several alternative approaches for extrapolating the design extreme stress range are then discussed. Following this, infinite life factors calculated using the simulation results are presented and the implications are discussed.

Background

For fatigue design of a component based on Canadian bridge design code (Clause 10.17), the stress range due to a single passage of the code truck, f_{sr}, multiplied by damage equivalence factor, λ_1, must be lower than the fatigue resistance, F_{sr}, of the component:

(1) $\lambda_1 \cdot f_{sr} < F_{sr}$

The damage equivalence factor, λ_1, is equal to 0.52 for all structural details except elements in bridge decks, and 0.62 for elements in bridge decks and expansion joints except floor beams and stringers. The fatigue resistance, F_{sr}, can be determined as follows:

(2) $F_{sr} = \left(\frac{\gamma}{N_c}\right)^{1/3}$

(3) If $F_{sr} = \left(\frac{\gamma}{N_c}\right)^{1/3} < F_{srt}$ then $F_{sr} = \left(\frac{\gamma'}{N_c}\right)^{1/5} \geq \frac{F_{srt}}{2}$

where F_{srt} is the constant amplitude threshold stress range of a component, γ and γ' are fatigue life constants, and N_c is the estimated number of truck passages during the design life of the bridge. Looking at Equations 1-3, a component with “infinite” fatigue life must satisfy the following:

(4) $\lambda_1 \cdot f_{sr} < \frac{F_{srt}}{2}$

To simplify the comparison in the rest of the paper, instead of evaluating the ½ factor that is multiplied by the fatigue threshold on the right side of Equation 4, an infinite life factor, λ_2, (similar to the method that is used in (AASHTO 2017) is defined as follows:

(5) $\lambda_2 \cdot f_{sr} < F_{srt}$

where λ_2 is the infinite life correction factor and is equal to 1.04 for all structural details except elements in bridge decks, and 1.24 for elements in bridge decks and expansion joints except floor beams and stringers.

Methods

For infinite life fatigue design, the peak stress range should be below the fatigue limit. This stress range is difficult to define, but fatigue tests have shown that when only a small percentage of cycles (e.g., 0.01%) exceed the fatigue threshold, crack propagation is negligible, and infinite fatigue life can be considered for practical purposes (Russo et al. 2016; Fisher 1993). In the North American bridge design codes (AASHTO 2017; CSA 2019), a component is said to have an infinite fatigue life if the stress range due to a single passage of the code truck (or the "nominal stress range") multiplied by a correction factor (e.g., $\gamma \cdot \Delta f$ in (AASHTO 2017)) is lower than a fatigue threshold (e.g., ΔF_{th} in (AASHTO 2017)). The purpose of the correction factor is to account for the difference between the nominal stress range and the peak stress range in the expected real traffic histogram, which governs infinite life design. In the current work, and based on the previous practice in North America, the 0.9999 percentile of the stress range histogram from a real traffic simulation (i.e., the stress range with a 1/10,000 reoccurrence rate) is taken as the peak stress range. This value is divided by the nominal stress range to determine the infinite life factor (λ_2) that should be used in the design code for infinite life design. On average, the stress range with a 1/10,000 reoccurrence rate occurs every 2.5, 10, 40, and 200 days in bridge elements on highways Class A, B, C, and D which have ADTTs of 4000, 1000, 250, and 50 respectively (CSA-S6). It should be noted that some highways have ADTTs much more than 4000 these days; stress ranges with a 1/10,000 reoccurrence rate can occur daily in bridge elements on these highways.

Traffic Simulation

The first steps for calculating the infinite life factor are similar to those described in (Hirt et al. 2006) for calibration of the fatigue correction factor (or “damage equivalence factor”) for finite life design. These steps were followed to obtain stress range histograms from real traffic data. To use this approach, the following information is required: the code truck model, the real traffic data, and influence lines either for a specific location on the bridge (e.g., if an assessment is being performed) or covering a broad range of possible cases (e.g., if the goal is to establish a correction factor for general use in a design code). The procedure employed to determine the infinite life factor is as follows (see Figure 2):

1) The code truck is passed over the influence line and the stress range is recorded. (Note: This stress range is referred to herein as the “nominal stress range”.)

2) The trucks in the real traffic database are randomly distributed between bridge lanes based on a defined distribution (e.g., 80% in Lane 1, 20% in Lane 2, etc.) and a random spacing is assumed between subsequent trucks in each lane, following a defined statistical distribution depending on traffic speed and volume.

3) The trucks in the real traffic database are passed over the same influence line and the stress peaks are recorded for the duration of the simulation period.

4) The rainflow cycle counting method in (Downing and Socie 1982) is used to generate stress range histograms from the stress peaks due to the real traffic recorded in Step 3.

5) The 0.9999 percentile of the stress range histogram is calculated (e.g., by fitting a curve through the histogram tail on appropriate probability paper) and divided by the nominal stress range to determine the correction factor for infinite life design.

Free moving traffic conditions were simulated in this work. According to (Bailey 1996), a shifted exponential distribution can be used to approximate the spacing between two subsequent trucks in free moving traffic conditions:

(6) $f_D(d) = \frac{V}{3600 \cdot s} \cdot \text{EXP}\left[-\frac{V}{3600 \cdot s} \cdot (d - 5.5)\right]$

where *s* is the speed of the traffic in meters/second, *V* is the traffic volume in trucks/hour, and *d* is the distance between subsequent trucks in meters. The trucks in each lane were displaced using the flow rate of that lane and then the truck in all lanes are passed over the influence line of the element of interest. To avoid passing the first trucks in each lane over the bridge together, a random distance from the bridge is assigned to the first trucks in each lane. The random distance from the bridge for the first trucks was also determined using Equation 6. The presence of the truck in adjacent lanes was assumed to be independent.

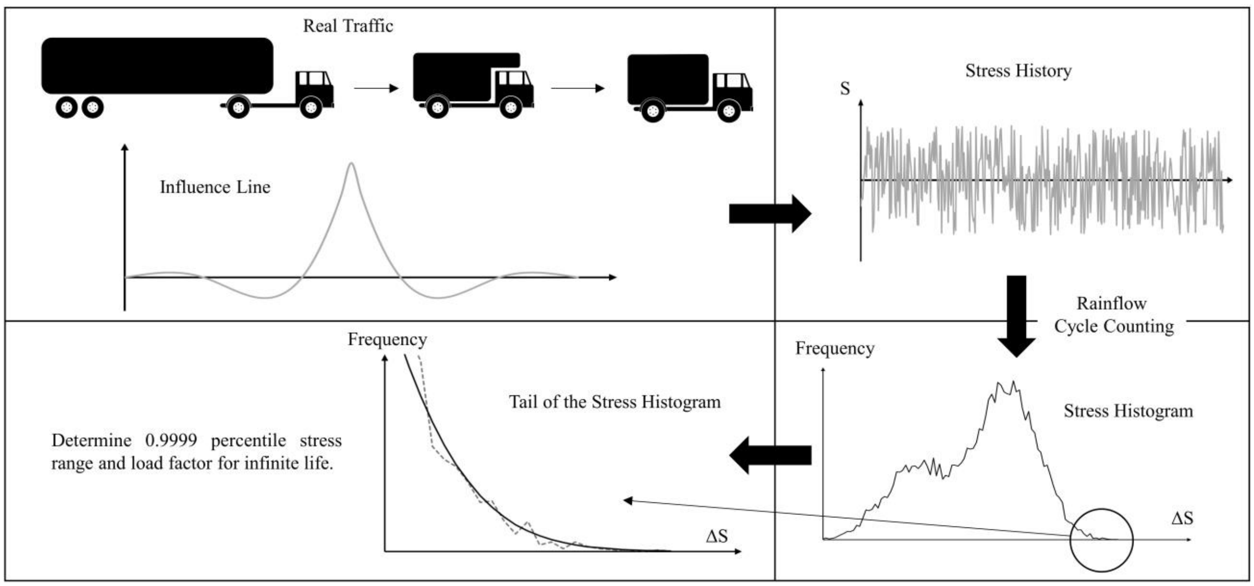

Figure 2. Determination of the infinite life factor.

Two real traffic databases from the Ontario Ministry of Transportation (MTO) obtained in 1995 and 2012 were used to simulate the traffic. These databases include axle weight and spacing data for each measured truck; the 1995 survey include 10,198 trucks and the 2012 survey include 45,192 trucks. The GVW histograms of the truck weights for MTO traffic databases are shown in Figure 3(a, b). The characteristics of CSA-S6 design trucks are shown in Figure 3(c, d). In Ontario, a special truck, CL-625-ONT as shown in Figure 3(c), is used for fatigue design. The gross weight of both design trucks is 625 kN. However, the Ontario truck has a heavier tandem axle which can affect the design of elements with short to medium influence lines. A DLA, dynamic load allowance, of 0.25 is assumed in CSA-S6 when three or more axles are on the bridge.

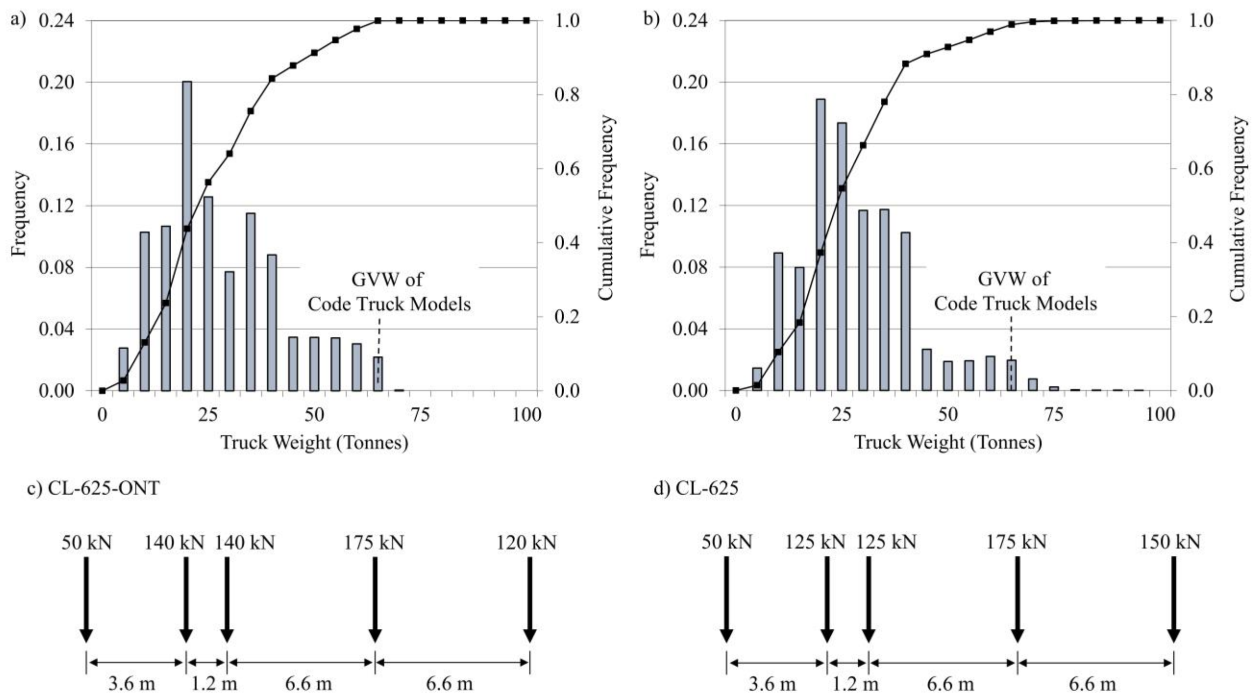

Figure 3. GVW histograms from MTO 1995 survey (a), and MTO 2012 survey (b), and code truck models used in CSA S6 (c, d).

In order to consider different bridge configurations, seven different influence lines were employed in the current study (see Figure 4): Positive bending moment at the mid-span for 1, 2, and 5 span girders (ps-m, p2tr-m, p5tr-m), negative bending moment at the mid-support for 2 and 5 span girders (p2tr-a and p5tr-a), and support reactions for 1 and 2 span girders (ps-r and p2tr-r). To cover a range of bridge spans, the following spans were used in this paper: 10, 15, 25, 50, 75, and 100 m. 1-lane (unidirectional) and 2-lane unidirectional and bi-directional traffic cases were investigated. For the bi-directional traffic, the trucks were randomly distributed between the lanes. In the current paper, illustrative results are presented for the unidirectional case with 80% of trucks in one lane and 20% in the other and the bi-directional case with a 50% probability of being in each lane. 80%-20% distribution of trucks in 2-lane unidirectional traffic was assumed due to the fact that typically more trucks are in the slow lanes in comparison with the fast lane (Algohi et al. 2018).

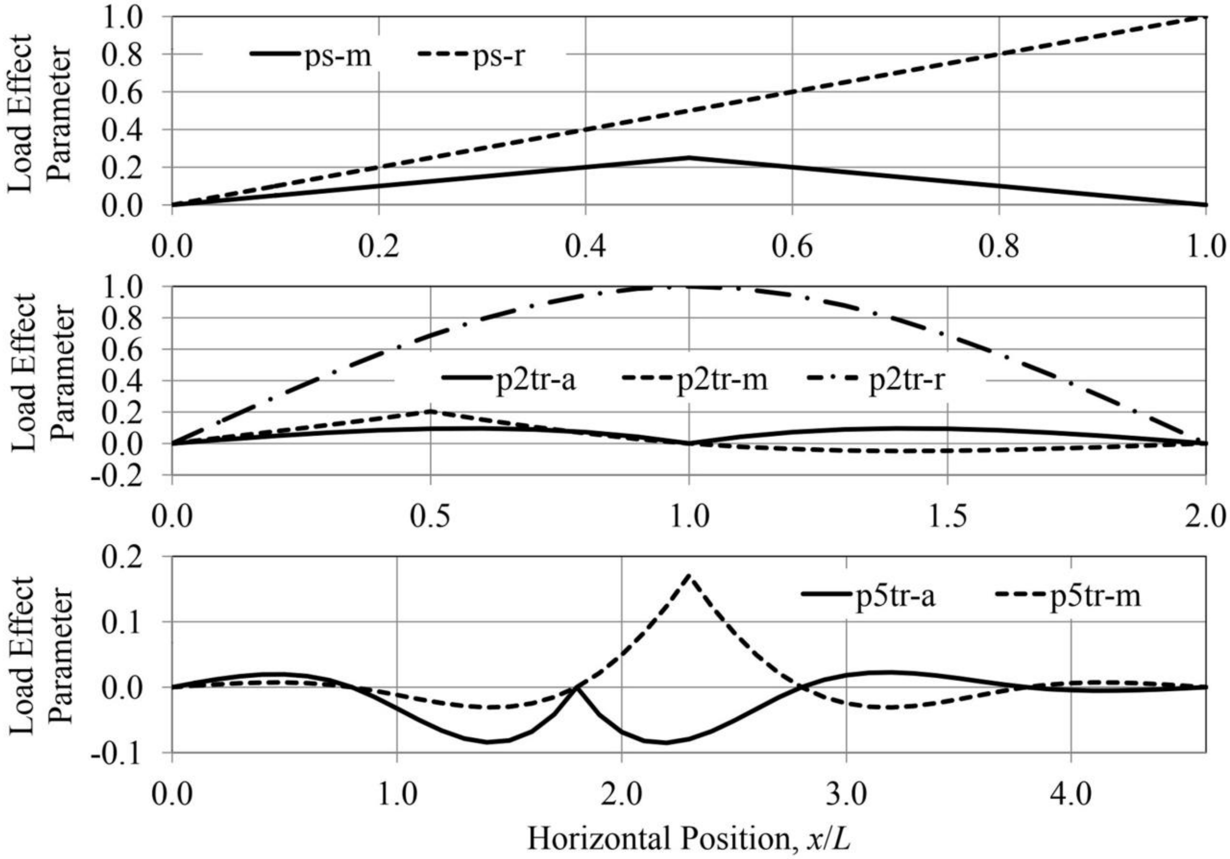

Figure 4. Investigated influence lines.

The dynamic load allowance (DLA), *IM* has been found in field studies to be related to several parameters including the bridge span, total static live load on the bridge, traffic speed, and road surface conditions (Kim and Nowank 1997; Laman et al. 1999; Schwarz and Laman 2001; Deng et al. 2015; Liu et al. 2019; Sjaarda et al. 2020). Simultaneous truck crossing events can significantly increase the total static live load on the bridge. To evaluate the effect of the assumed DLA model on the infinite life factor, two models were used: a constant DLA equal to the code-prescribed value and a variable DLA model (See Figure 5), applied to the real traffic, which considers the effect of the total static live load on the bridge at each point in time (Ludescher 2003). The variable model is adjusted to give a DLA of 0.25 at the GVW of the code truck, 625.0 kN.

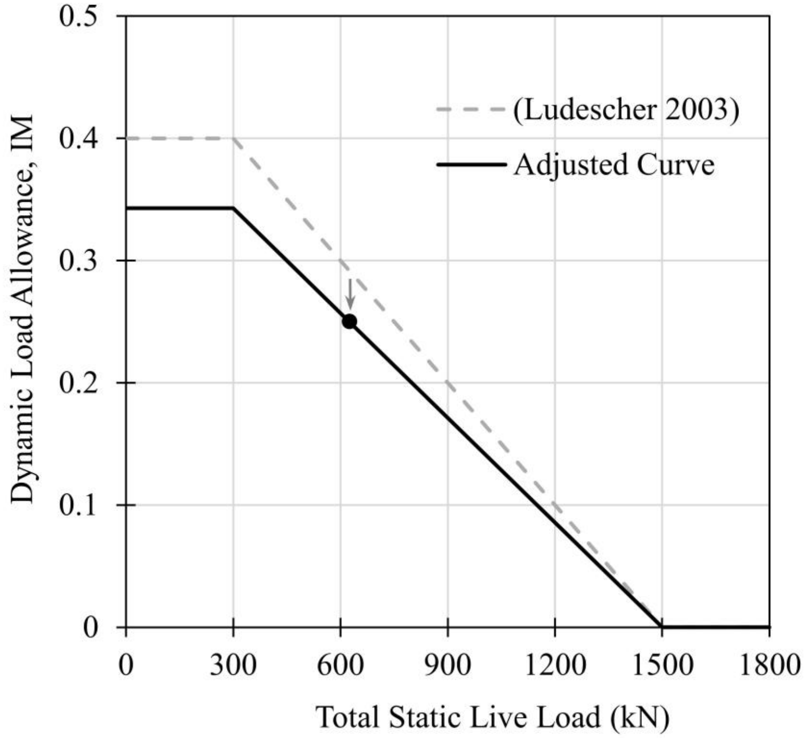

Figure 5. Variable DLA models.

Both "one at a time" and simultaneous truck crossing cases were evaluated in this work. Non-truck traffic is not included in the simulation. Its primary effect is to increase the spacing between subsequent trucks. This effect was considered by using several truck-only flow rates, similar to those used in (Walbridge et al. 2013). Specifically, the following "steady-state" flow rates were employed: 0.047, 0.1, and 0.2 veh/s. Based on time of day and location, the actual traffic flow rates on a bridge can vary significantly over time. In the current study, after investigating the steady-state flow rates, variable hourly truck flow rates for Highways 401, 403, and 6 in Ontario are employed to evaluate the effect of variable – real flow rates on infinite life factors (See Figure 6). These hourly truck traffic flow rates are based on the data collected between 2006 and 2008 (MTO 2008).

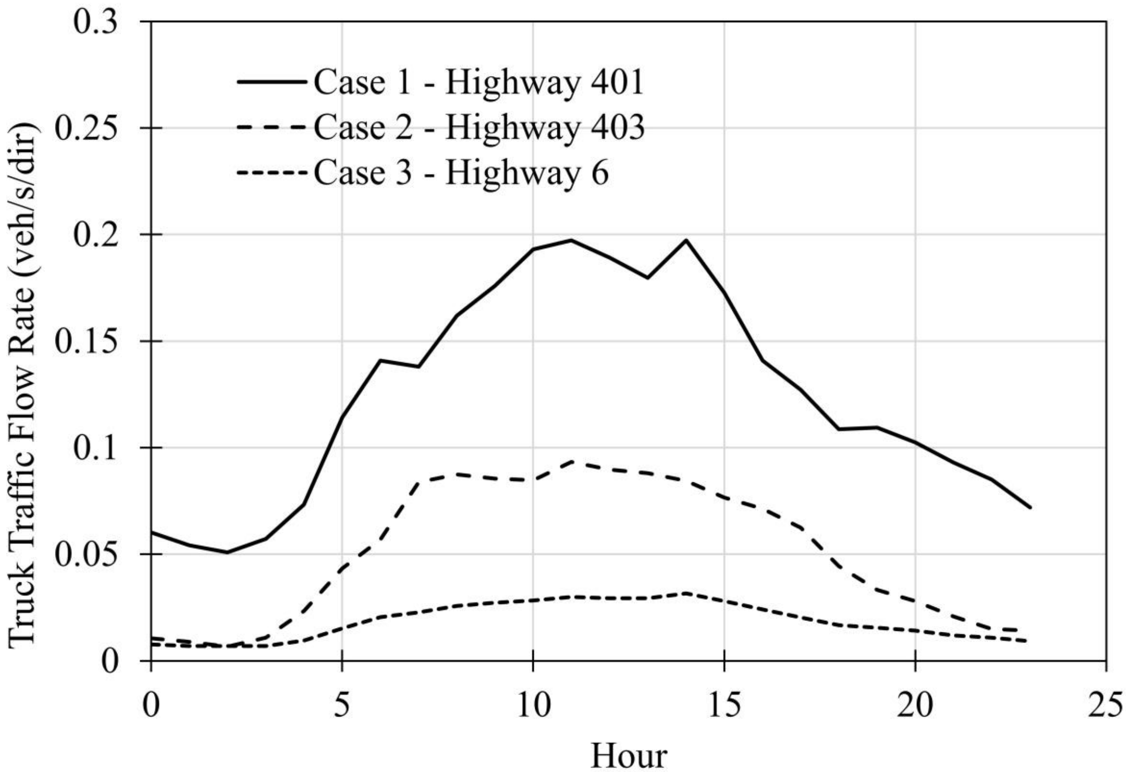

Figure 6. Hourly truck traffic flow rates based on data available from (MTO 2008).

Calculation of Stress Range with 1/10,000 Reoccurrence Rate

In order to determine the 0.9999 percentile stress range, the tails of the histogram outputs from the real traffic simulation were fitted against two distribution types, Normal and Weibull, in order to determine the extent to which the assumed distribution type affects the analysis. Only the upper tails of the histograms were of interest for distribution fitting. For each distribution type, data from the upper 20% range (80 to 99.9 percentile) and upper 10% (90 to 99.9 percentile range were fitted, in order to determine the extent to which the definition of the tail size affects the analysis. The top 0.1 percentile was not included in the curve fitting process, as the values significantly affected the linearity of the fitted curve.

To determine the best-fit distribution parameters, the slope (*m*) and y-intercept (*b*) of each fitted linear curve were found. Linear interpolation/extrapolation was then used to determine the stress ranges associated with the 0.9999 percentile. In the laboratory experiments originally used to suggest the 0.9999 percentile guideline, a “per cycle” definition of this percentile was assumed

(Russo et al. 2016). For the current study, it was debated whether this definition should be applied (a "per cycle" basis) or a "per truck" basis, the former being the definition used in the fatigue tests, and the latter being arguably more rational, given that the 0.9999 percentile stress range can be heavily affected in an in-service variable amplitude loading analysis by the way in which small cycles are counted (or ignored). Initially, both definitions were considered in order to investigate the effect of this assumption on the analysis results.

The equations for the lines of best fit for the Normal and Weibull distributions are as follows:

(7) $\Delta f_p = m_{normal} \cdot \Phi^{-1}(p) + b_{normal}$

(8) $\mathrm{LN}\big(\Delta f_p\big) = m_{Weibull} \cdot \Big(\mathrm{LN}\big(-\mathrm{LN}(1-p)\big)\Big) + b_{Weibull}$

where Φ^{-1} is the inverse of the standard normal distribution and p is the percentile (e.g., 0.9999). The histogram obtained from the real traffic simulation was established for the fixed number of trucks employed in the simulation. Thus, in calculating the 0.9999 percentile stress range event, it had to be normalized, as each truck typically causes multiple stress ranges. In the calculation of the 0.9999 percentile truck event, the corresponding stress range event needs to be obtained by considering the average number of cycles per truck. In other words, the 1/10,000 truck event corresponds with the stress range event having a reoccurrence rate of 1 / (10,000·average number of cycles per truck). It should be noted that a third case was also considered, wherein the average number of cycles per truck assumed by the code was used (N_d in (CSA 2019)). However, this assumption did not prove to have a major significance on the extreme stress range calculation, and the results for this case are therefore not presented here.

Results

The infinite life correction factor, λ_2, was first obtained using the MTO 2012 survey data and Ontario Truck (note: all the results in this paper are obtained using the CL-625-ONT truck). Sample results are shown in Figure 7 for two traffic conditions: one truck at a time passing over the influence line and 1-lane traffic at the high flow rate of 0.2 veh/s. Envelopes (bounded by the solid and dashed lines) are plotted in this figure of the results for the seven investigated influence lines vs. bridge span. Looking at this figure, a number of observations can be made. First, the results are similar, regardless of whether the 1/10,000th stress range or the 1/10,000th truck is used for the extreme event definition. Although not shown here, it was similarly found that assuming a normal vs. a Weibull distribution and using the upper 10% of the data vs. the upper 20% did not affect the results significantly, with the exception that significant differences (greater than 1-2%) could be seen when the Weibull distribution and upper 20% of the data were used in combination. Qualitatively, it was seen that the normal distribution appeared to provide a better fit of the simulation data. On this basis, subsequent analyses focused on the case of the 1/10,000th truck event and a normal tail assumption using the upper 10% of the simulation data.

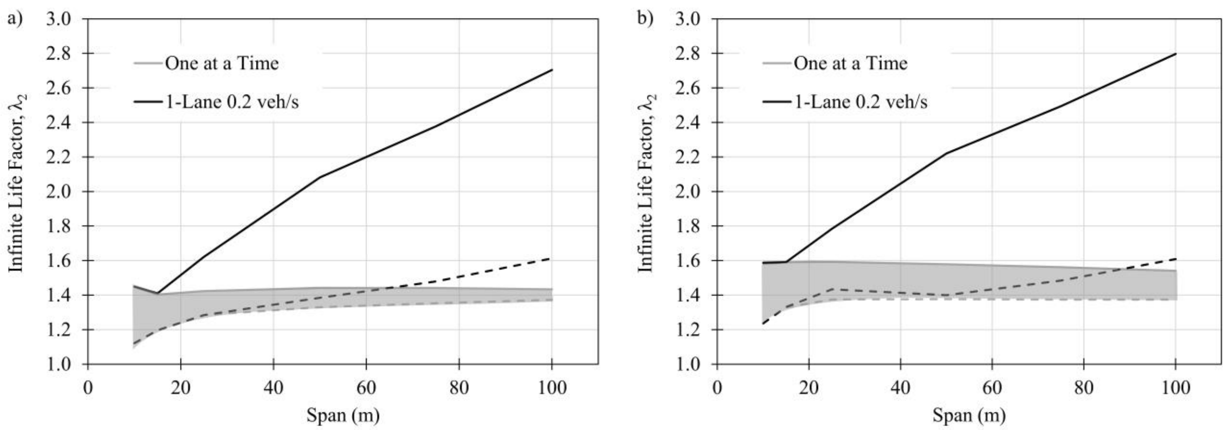

Figure 7. Results for MTO 2012 traffic 1/10,000th cycle (a) vs. 1/10,000th truck (b).

Looking at Figure 7, it can also be seen that the infinite life factors increase significantly when a high-volume traffic simulation with simultaneous vehicle crossings is considered (due to vehicles following each other closely). It is unlikely bridges will see such flow rates for extended periods – 0.2 veh/s corresponds with 17,280 veh/day, which would be extraordinarily high for a single lane road, although short periods at flow rates this high are considered to be possible, as discussed in (Walbridge et al. 2013).

Lastly, it can be seen in Figure 7 that even in the one at a time analysis case, the average infinite life factor obtained in the simulation is considerably higher than the code-prescribed value of 1.04. Intuitively, this is not surprising, since the largest truck in the database is 94.4 tonnes, which is greater than the GVW of the 62.5 tonnes code truck by a factor of 1.51 (which is close to the mean correction factor value on the vertical axis for the one at a time analysis in Figure 7(b)). In the 1995 MTO survey, the heaviest truck is 69.5 tonnes (69.5 / 62.5 = 1.11), which suggests a simulation result with an average factor closer to 1.04 should be more likely using this data. In Figure 8(a), the "one at a time" simulation results for different influence lines for MTO 1995 database are compared. Here it can be seen that although the infinite life factor is lower when the 1995 database is used, in comparison with the results for MTO 2012 in Figure 7(b). However, it is still quite a bit higher than 1.04. In order to understand the reason for this, the tail fitting analysis was inspected for several typical cases obtained using the 1995 data. A typical result is shown in Figure 8(b). Looking at this figure, it can be seen that in the case of the 1995 database, the extrapolated ("best fit") curve (to the required standard normal percentile of 3.8 – 4.1) appears to overestimate the extreme stress range significantly. Looking at the GVW histogram for this database in Figure 3(a), it can be seen that the upper tail of the GVW histogram drops suddenly on the high end, whereas in the 2012 database, more heavy trucks were captured, resulting in a better fit with the assumed normal distribution tail.

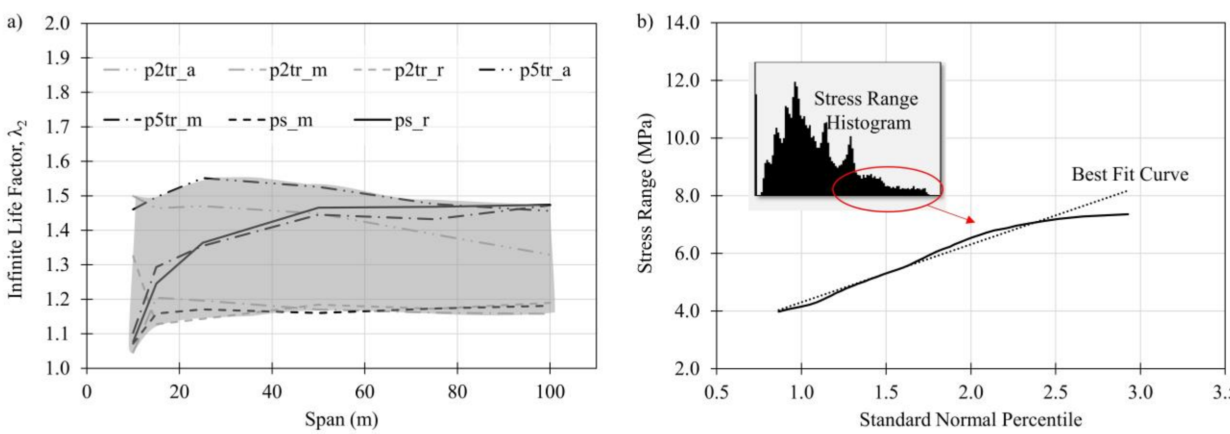

Figure 8. One at a time crossing results for MTO 1995, 1/10,000th truck (a) and example of normal tail fit for MTO 1995 traffic, p2tr_a span = 100 m analysis case (b).

Simultaneous truck crossing effects

Figure 9(a, b) shows average infinite life factor curves (for all seven influence lines) obtained using MTO 1995 and 2012 databases for a range of flow rates for both 1-lane and bi-directional traffic cases. Comparing these cases, it can be seen that higher infinite life factors result in the case of 2-lane bi-directional traffic. This is due in part to the fact that the traffic volume is doubled in the 2-

lane case, but even when similar volumes are compared (e.g. 1-lane 0.2 veh/s with 2-lane bi-directional 0.1 veh/s/lane) the factors are higher in the 2-lane case, since simultaneous crossings are possible not only when vehicles travel close to each other but also due to vehicles crossing the bridge at the same time while travelling in opposite directions. Note that in this analysis, it is assumed that the traffic in both lanes for the multi-lane cases has the same influence on fatigue damage. This might be an appropriate assumption for example for a fatigue detail on a single box-girder bridge (with high torsional stiffness) carrying multiple lanes of traffic. It might be a conservative assumption (and perhaps excessively so) for a fatigue detail on a multi-girder bridge with a flexible deck, where each girder carries load primarily from the traffic directly over it. In this case, the results for the 1-lane analysis might be more relevant.

An interesting point in this figure is that the simultaneous truck crossing results based on both databases are very close for elements with long influence lines. However, for elements with short influence lines, simultaneous truck crossing results for MTO 2012 database are higher than the results for MTO 1995 database. This is due to the fact that extreme stress ranges for elements with long influence lines are governed by the total weight of multiple random trucks that can be on the bridge at the same time. The total weight of multiple random trucks can be very close when two different databases are employed. However, for elements with short influence lines, the simultaneous truck crossing effects are limited, and the results are mainly governed by individual truck weights in the databases, therefore, lower results are expected for MTO 1995 database.

Figure 9(c, d) shows the results of MTO 1995 and 2012 simulations with 2-lane unidirectional traffic with an 80-20 split of the trucks between the two lanes, as might be expected for example if most trucks are driving in the “slow” far-right lane. These results are compared with 1-lane and 2-lane bi-directional results presented previously. This comparison shows the 2-lane unidirectional 80-20 split case falling between the 1-lane and the 2-lane bi-directional cases, suggesting that these cases represent bounds on the infinite life factor for a given traffic volume.

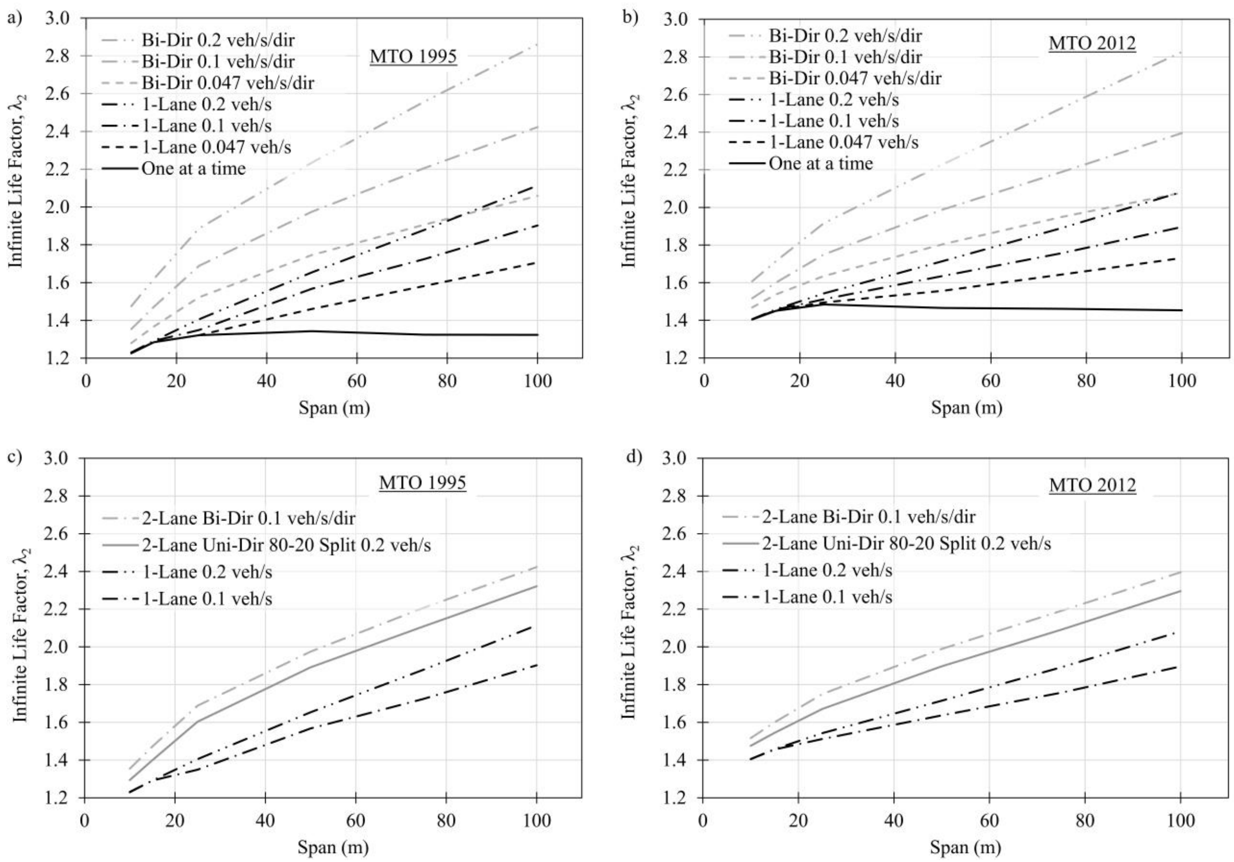

Figure 9. Average 1-1ane and 2-lane Bidirectional results for MTO 1995 (a) and MTO 2012 (b), and 2-lane unidirectional results for MTO 1995 (c) and MTO 2012 (d) traffic (1/10,000th truck).

In order to isolate the effects of simultaneous truck crossings from other factors such as the choices made in defining the design extreme stress range event (e.g., distribution type, percentage of the upper tail to use in curve fit), a new factor (α) is proposed to isolate and account specifically for the simultaneous truck crossing effects. This factor is taken simply as the ratio between the λ_2 values obtained in the simultaneous crossing analysis with that obtained in the corresponding "one at a time" analysis. A practical application of the α factor would consist of a designer multiplying λ_1 in Equation 4 by α in order to magnify it to appropriately account for simultaneous crossing effects. Results are plotted in Figure 10(a, b) for both databases. Looking at this figure, it can be seen that the α values follow similar trends, with the worst case of high-volume traffic in a long span bridge resulting in a α value of around 1.5 for 1-lane traffic and 2.1 for bi-directional traffic, and the α factor diminishing to unity for lower traffic volumes and shorter bridge spans. It should be noted that, while the infinite life factors for both databases are similar for elements with long spans, higher simultaneous truck crossing factors, α, are calculated for MTO 1995. This is because of lower one at a time crossing results for MTO 1995 databases. MTO 2012 database includes heavier trucks and those heavy trucks result in higher factors for one at a time crossing in comparison with MTO 1995 database. However, the simultaneous crossing results for elements with long spans for both databases are similar (see Figure 9). Therefore, the proposed factor, α, which relates the simultaneous crossing results to the one at a time crossing results, is higher for MTO 1995 database.

It could be argued that the presented simulation-based analysis is erring on the conservative side for a number of reasons. One of these is the well-known trend of the DLA decreasing with an increase in GVW or the total static live load of the trucks on the bridge at a given time. In order to consider this possibility in a simple way that does not involve a full-blown dynamic analysis, the adjusted model from (Ludescher 2003) was employed and the α factor was recalculated with the code DLA (IM) applied to the code truck and the variable DLA (Adjusted curve in Figure 5) applied to the real traffic and varying at each step of the simulation depending on the continuously varying total static live load. The results of this simulation are plotted in Figure 10(c, d). In this figure, it can be seen that the overall trends are similar and the α factor is still greater than unity, although it decreases somewhat with the variable DLA model employed.

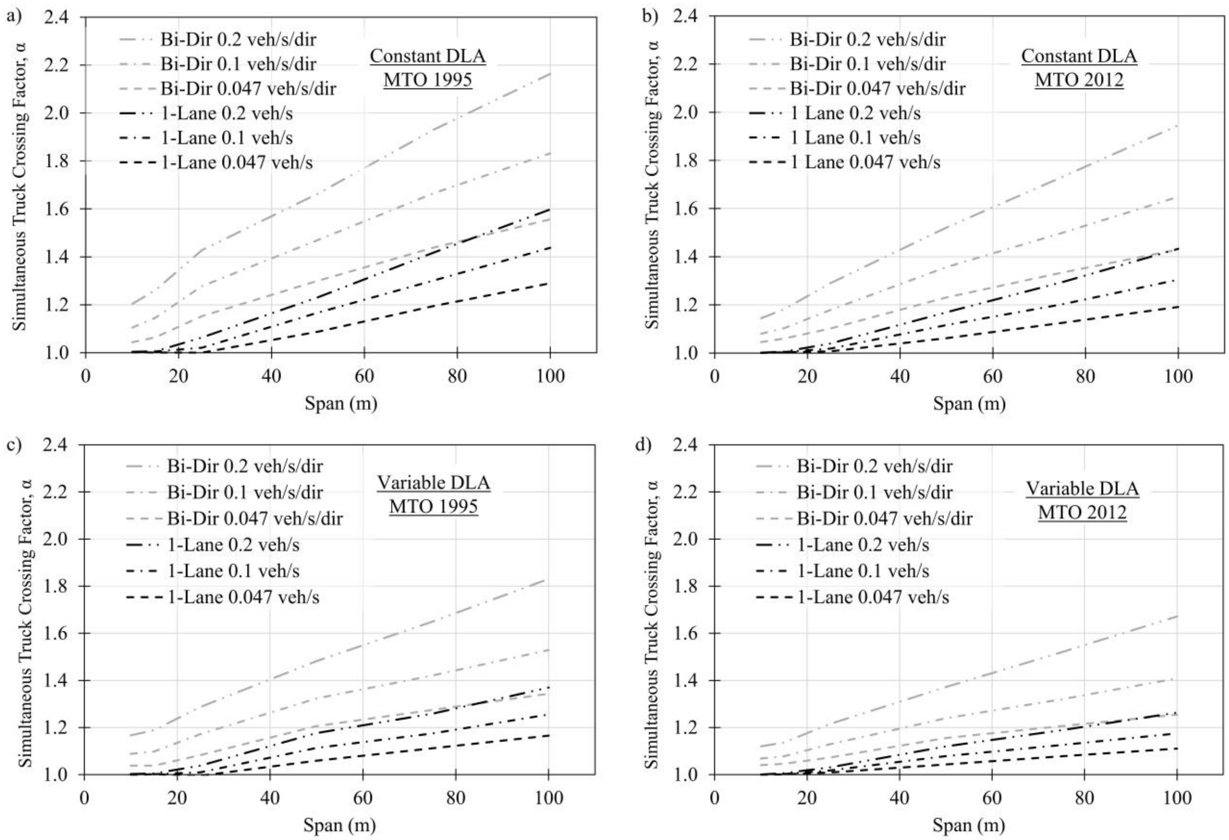

Figure 10. Proposed α simultaneous crossing effects for constant and variable DLA models.

To evaluate the effect of variable flow rates, the α factors are calculated using hourly flow rates presented in Figure 6 for Highways 401, 403, and 6 in Ontario. Note that the previously investigated 1-lane and 2-lane unidirectional 80-20 split cases are evaluated here, so only the variable flow rates from these locations are used, primarily for illustrative purposes, and not other aspects of the roadway geometry, such as the actual number of lanes. For variable flow rate simulations, one day was divided into 24 1-hour blocks and the corresponding flow rates from Figure 6 were used. As can be seen in Figure 11, the simultaneous truck crossing factors are found to greatly depend on the location of the bridge and are much higher for highways with high hourly flow rates. However, comparing these results with the constant flow rate results shows that using the maximum value of flow rate to determine the simultaneous truck crossing effect can be conservative. It should be noted that considering the whole traffic of one direction in one lane is conservative for elements that carry the load of one lane. For the case of a 2-lane unidirectional 80-20 split, it is assumed that the element carries the loads from both lanes. The results for 2-lane unidirectional traffic are higher, as multiple trucks can be present on two lanes simultaneously.

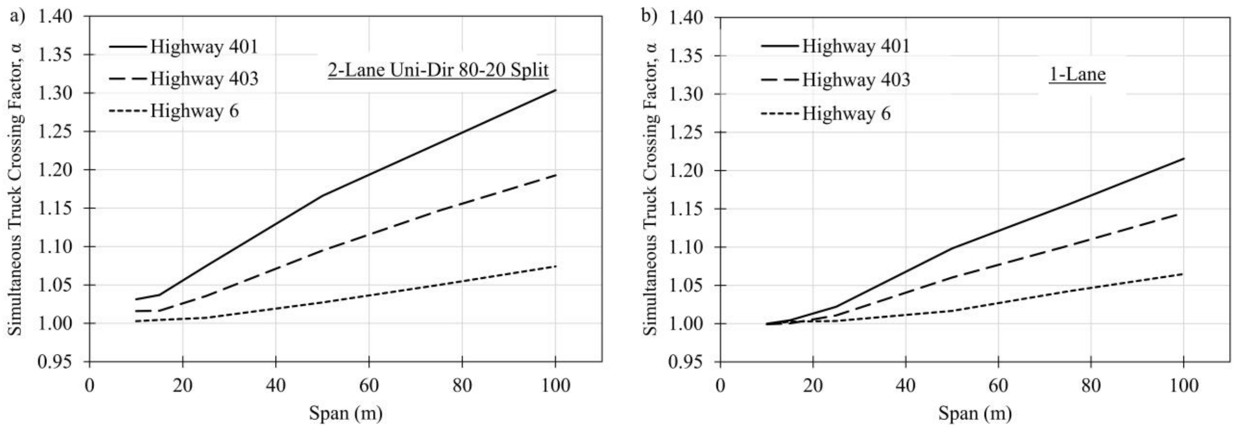

Figure 11. α obtained using variable DLA model based on MTO 2012 databases, employing variable flow rates.

Figure 12 compares constant flow rate simulation results with variable flow rate simulation results for Highway 401. It can be seen that employing a constant flow rate equal to the maximum flow rate for this highway, 0.2 veh/s, is conservative. The constant flow rate simulations were also performed using two different average flow rates. First, an hourly average flow rate, $\bar{V}$, was used:

(9) $\bar{V} = \frac{\sum v_i}{24}$

where v_i is the hourly flow rates (veh/s) shown in Figure 6. As can be seen in Figure 12, the results based on this average are lower than the variable flow rate simulation results for Highway 401. It can be argued that this hourly average is not appropriate because more trucks are crossing the bridge during the hours with high flow rates. Therefore, a weighted average flow rate, $\bar{V}_w$, was employed:

(10) $\bar{V}_w = \frac{\sum N_i \cdot v_i}{\sum N_i}$

where N_i is the number of trucks that cross the bridge in each hour and can be determined as follows:

(11) $N_i = 3600 \cdot v_i$

Looking at Figure 12, it can be seen that the results based on the weighted average flow rates are very close to results for the variable flow rates; the results are much closer for 1-lane simulations. This constant weighted average flow rate approach was then used for Highways 403 and 6. It can be seen that the results based on the weighted averages are also close to the variable flow rate simulation results for these highways.

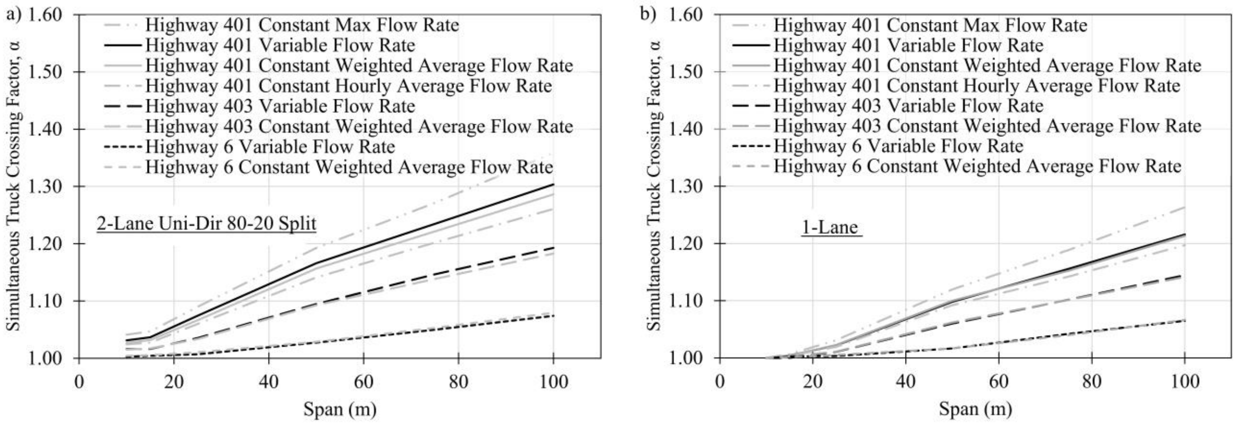

Figure 12. Comparing α obtained using variable and constant flow rates based on MTO 2012 databases.

Conclusions

Based on the analysis results presented in this paper, the following conclusions are drawn:

- Simultaneous truck crossing events can have a significant influence on the correction factors associated with infinite life design for roadway bridges, especially for bridge elements with long influence lines.
- When simultaneous truck crossing events are considered in the simulation, the effect of employing different real traffic databases on the infinite life factors for elements with long influence lines is minimal. However, for elements with short influence lines, the GVWs of the trucks in the database are a critical parameter.
- The effect of simultaneous truck crossings can be accounted for using a factor α, which varies as a function of the bridge span and traffic type and flow rate.

- The influence of simultaneous truck crossings is reduced when the effect of total static live load on the DLA is considered. However, it can still be significant in certain cases.

- The infinite life factor results based on the variable hourly truck traffic flow rates for three highways in Ontario show the significant effect of flow rates/location of the bridge on the infinite life factors.

- The variable flow rate results were compared with three different constant flow rates results. It was found that weighted average flow rate simulation results are very close to the variable flow rate simulation results, especially for 1-lane cases.

The α factors presented in this paper are believed to provide bounds and offer a practical means for adjusting the infinite life design factors to account for simultaneous crossing effects, either with a general code-prescribed adjustment of this factor for all structures, or with an adjustment only applied by the designer only for structures where it is believed that simultaneous truck crossing effects may lead to unconservative designs using the existing provisions.

A number of limitations of the presented analysis and possible future work avenues are worth mentioning. First, the investigated constant traffic flow rates represent extremes, which can occur during short periods, but are unlikely to be seen on a continuous basis in actual bridges. For this reason, it might make sense to employ the proposed correction factor (α) on a pro-rated basis, considering different traffic flow rates over the course of a typical day. This may be difficult to do in practice though, in particular for a bridge that hasn't been built yet. For many bridge types, it seems the effect of simultaneous truck crossings can be ignored with little consequence. This includes short span bridges, and in particular those consisting of multiple girders, where each girder is primarily carrying the load from only one lane of traffic. This likely covers the vast majority of bridges that most highway authorities will have in their inventory. Expansion joints are an example of elements with very short influence lines that do not get affected by simultaneous truck crossing events. Perhaps the most valuable takeaway from this analysis is that the current correction factors for infinite life design have possible limitations, which should be taken into account when designing unusual structures, including longer span bridges and bridges consisting of a single element (e.g., box girders) supporting traffic from several lanes. Bridges on very high-volume roads or roads leading to bottlenecks such as toll booths and border crossings also likely fall into this category. Thus, congested or “stationary” traffic cases are another recommended area of future study.

Acknowledgments

Martin Haalstra and Alexander Au at the Ministry of Transportation of Ontario (MTO) are sincerely thanked and acknowledged for their insight and enthusiastic support and for providing access to the MTO 2012 survey data used in this study.

Competing interests

The authors declare there are no competing interests.

Data availability

MTO 1995 and 2012 traffic databases used during the study were provided by a third party. Direct request for these materials may be made to the provider as indicated in the Acknowledgements.

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