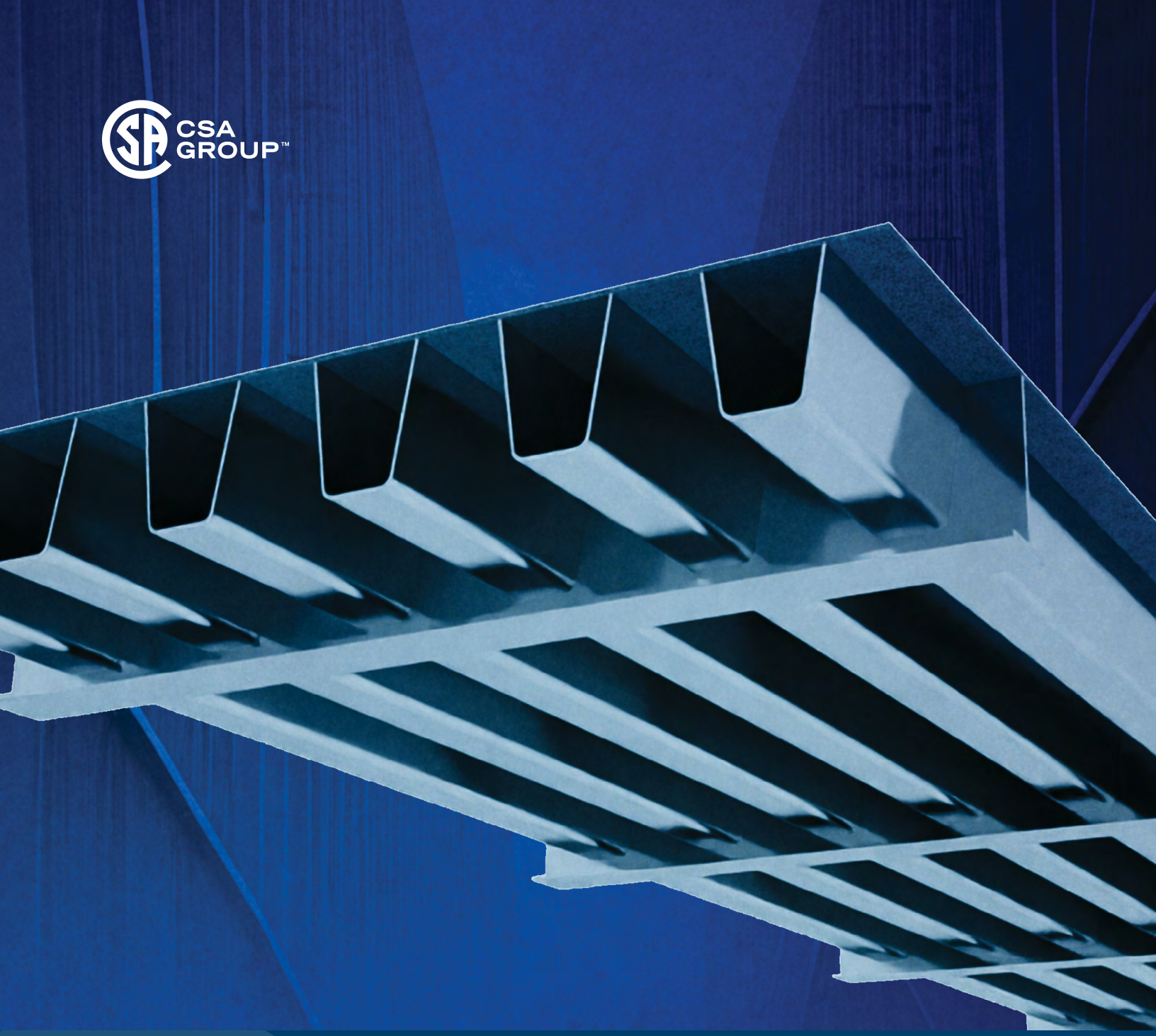


STANDARDS RESEARCH

Design of Orthotropic Steel Bridge Decks

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Executive Summary

Orthotropic bridge decks are efficient, lightweight steel deck systems, typically fabricated by welding "ribs" to the underside of a flat steel plate. The potential benefits of orthotropic steel deck (OSD) systems are compelling. For example, in the early 1990s, the deteriorating concrete deck on Montreal's Champlain Bridge was replaced with an orthotropic steel deck that weighed 25% less than the original deck.

In the last code revision cycle (2019), the orthotropic bridge deck design provisions in CSA S6:19, *Canadian Highway Bridge Design Code,* were replaced with references to the American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications* to capture the current state of knowledge and to address needs in the CSA S6 provisions until more Canadian-specific guidance could be developed.

As a result, CSA S6:19, Clause 10.16 requires that the design of OSDs satisfy the requirements of Clause 9.8.3 of the AASHTO specifications while still using loads and factors from CSA S6:19 for the serviceability, fatigue, and ultimate limit states (SLS, FLS, and ULS). Due to differences in the applied factors and loads, there are instances where using this approach is not only difficult but may not be appropriate from a structural safety or economic perspective. For example, in fatigue design, it is not clear when to switch from the AASHTO provisions for analysis to the CSA provisions to account for differences in Canadian truck traffic. Similarly, the current approach may over-penalize Canadian bridges because it requires using the AASHTO deflection limits in conjunction with the heavier Canadian design truck. These issues raise concerns that may discourage designers in Canada from using orthotropic deck solutions for new projects.

This report summarizes the results of a study that investigated these issues and presents recommendations for revisions to CSA S6 for the 2025 code cycle. The main outputs of this research include a literature review, an industry survey, applications of the existing design provisions to a sample deck system, a short investigation of Canadian truck weight data from two provinces, and recommendations for revised provisions to be included in the 2025 edition of CSA S6.

Industry experts, including members of the Project Advisory Panel, with prior experience in orthotropic deck design, reported that high fabrication cost, long-term issues with the wearing surface, and fatigue cracking are the most common factors that dissuade designers from considering OSDs as a design alternative. Furthermore, experts identified several challenges experienced during the design and construction of previous decks, including ensuring an adequate penetration level for the rib-to-deck plate welds with no blow-through, difficulty selecting appropriate deck dimensions, confusion in navigating between the CSA and AASHTO provisions, and satisfying the imposed deflection limits. Designers and fabricators who participated in the industry survey also shared advice on best practices for designing optimum OSD solutions from a cost and performance perspective.

Deflection and fatigue analyses were also conducted for this study. Deflection analyses of a sample deck system showed that two-way deflection is approximately 40% higher than one-way deflection. These analyses also confirmed that the current approach results in heavier deck designs for Canadian bridges because it requires using the AASHTO deflection limits in conjunction with the heavier Canadian design truck. However, this requirement may be justified, given the apparent presence of heavier trucks in the available Canadian highway traffic data. A fatigue analysis was performed using AASHTO Level 3 design for the same sample deck system. It involved application of the hot-spot stress method using a finite element (FE) analysis. A comparison of the weld toe stresses using the CSA and AASHTO trucks and load factors showed that the Canadian design truck and load factors result in stresses that are 55% and 35% higher for finite and infinite fatigue life design, respectively. As a result of this exercise, instances were identified where it may not be clear to the designer when to use factors or provisions from CSA S6 or the AASHTO specifications. Gaps in the AASHTO procedure, such as specific instructions on how to perform notch stress analysis of the rib-to-deck plate weld root, were also identified and solutions were proposed where possible. This report concludes with proposals for revisions to CSA S6 and for future research to improve CSA S6 beyond 2025.



OSDs are efficient, lightweight deck systems that lend themselves well to applications where minimization of the dead load (i.e., self-weight) is important.

1 Introduction

Orthotropic steel decks (OSDs) are structural systems consisting of a thin steel plate supported by floor beams, typically perpendicular to the traffic direction, with longitudinal, often U-shaped, ribs stiffening the deck plate. Due to their configuration, OSDs exhibit different properties in the transverse and longitudinal directions. OSDs are efficient, lightweight deck systems that lend themselves well to applications where minimization of the dead load (i.e., self-weight) is important. Furthermore, if properly designed, they require little maintenance throughout the structure's service life and are therefore appropriate for applications where access to the deck would be difficult. As a shop fabricated product, OSDs can be produced with tighter quality control. Deck configurations have minimal joints, which decreases the risk of leaking and exposure of the superstructure to de-icing salts, resulting in advantages from a durability perspective. Finally, OSDs enable modularization and can therefore play an important role in rapid construction [1] and deck replacement with minimal disruption (e.g., during night closures) for the rehabilitation of existing bridges.

CSA S6:19, *Canadian Highway Bridge Design Code* [2], Clause 10.16 requires that the design of OSDs satisfies the requirements of Clause 9.8.3 of the American Association of State Highway and Transportation Officials (AASHTO) *LRFD Bridge Design Specifications* [3] while still using loads and factors from CSA S6:19 for the serviceability, fatigue, and ultimate limit states (SLS, FLS, and ULS). Due to differences in the applied factors and loads, there are instances where using this approach is not only difficult but may not be appropriate from a structural safety or economic perspective. For example, in fatigue design, it is not clear when to switch from the AASHTO provisions for analysis to the CSA provisions to account for differences in Canadian truck traffic. Similarly, the current approach may over-penalize Canadian bridges because it requires using the AASHTO deflection limits in conjunction with the heavier Canadian design truck. These issues may discourage designers in Canada from using orthotropic deck solutions for new bridge construction projects or for projects involving the rehabilitation of existing bridge structures.

On this basis, an investigation was undertaken with the following objectives:

- to review the state of knowledge on OSD design with a focus on the loading, analysis methods, and fatigue performance of the rib-to-deck plate weld;
- to perform preliminary structural analyses to test the implementation of the CSA S6:19 design approach and identify issues to be resolved; and
- to produce a report that includes a summary of findings, recommendations for CSA S6:25, and areas of improvement for future research and future editions of CSA S6.

This report presents key aspects of the work that was conducted to address these objectives. Section 2 provides a review of the state of knowledge in research on OSDs, with a focus on the two biggest issues impacting the durability and success of these structural systems: fatigue and the wearing surface. Section 3 summarizes the findings from a survey of industry experts, which include the identification of specific design challenges faced by practitioners, as well as past experiences and best practices for designing optimum OSD solutions from a cost and performance perspective. Section 4 describes the structural analysis methods used to study the deflection and fatigue performance of a sample deck system. Section 5 presents the results of the analyses performed using the methods detailed in Section 4. Section 6 presents a study of available gross vehicle and tandem axle weights in Ontario and Alberta. The report concludes with a summary of proposed changes for the 2025 edition of CSA S6 and recommendations for future research.

2 Literature Review

2.1 Background

OSDs have been in use internationally since the mid-1900s due to the advantages they offer, such as their light weight and durability [1]. However, further implementation of OSDs has been hampered by reported negative experiences, particularly regarding premature failure of wearing surfaces and fatigue damage, which often occurs at the rib-to-deck joint welds and the complex fatigue details around the floor beam-to-rib connections, the causes of which require further elaboration.

Without adequate protection, the deck plate surface of an OSD system would be susceptible to corrosion and offer poor skid resistance for moving vehicles. It would also contain surface irregularities due to the fabrication process [4]. These issues are addressed by applying a wearing surface on top of the deck plate, which provides corrosion protection, skid resistance, and covers the surface irregularities [4], [5]. However, poor performance of the wearing surface compared with more conventional deck systems is known to be a recurring problem with orthotropic steel bridge deck systems.

Fatigue cracking of the decks — typically at the welds — occurs because the deck ribs are often connected to the deck plate through a single-sided partial joint penetration (PJP) weld. From a fabrication perspective, this welding technique has advantages over full penetration welds, but it does not generally perform as well under fatigue (i.e., cyclic) loading because the weld root is essentially a sharp notch, or stress concentration. Analyzing the stresses in the critical locations of the weld and designing and fabricating the complex fatigue details around the floor beam-to-rib connections can also be challenging. The fatigue performance of the welds in OSDs is therefore a common concern [6-23].

2.2 General Knowledge on OSD Design

A wealth of information based on research and industry experience can be found in guidelines on OSD design, such as the manual produced by the US Federal Highway Administration (FHWA) [1]. The manual includes a survey of OSD projects with details such as deck plate and rib dimensions, which can be useful for preliminary sizing before detailed design verifications are performed. Typical ranges for these dimensions can be summarized as follows, and are labelled in a schematic in Figure 1:

- Spacing of floor beams: 3–6 m (A)
- Deck thickness: 12–16 mm (B)
- Rib depth: 203–356 mm (C)
- Upper rib width: 292–387 mm (D)
- Rib thickness: 6–11 mm (E)
- Lower rib width: 165 mm (F)
- Center-to-center rib spacing: 600 mm (G)
- PJP welds between ribs and deck plate: 70–80% penetration (H)

Another important variable in the design of OSDs is the shape of the stiffeners/ribs. A variety of shapes (e.g., trapezoidal, U-shaped, V-shaped, T-shaped, flat) have been used [1]. The FHWA manual provides a flow chart suggesting a sequence for the design verification that includes the following steps [1]:

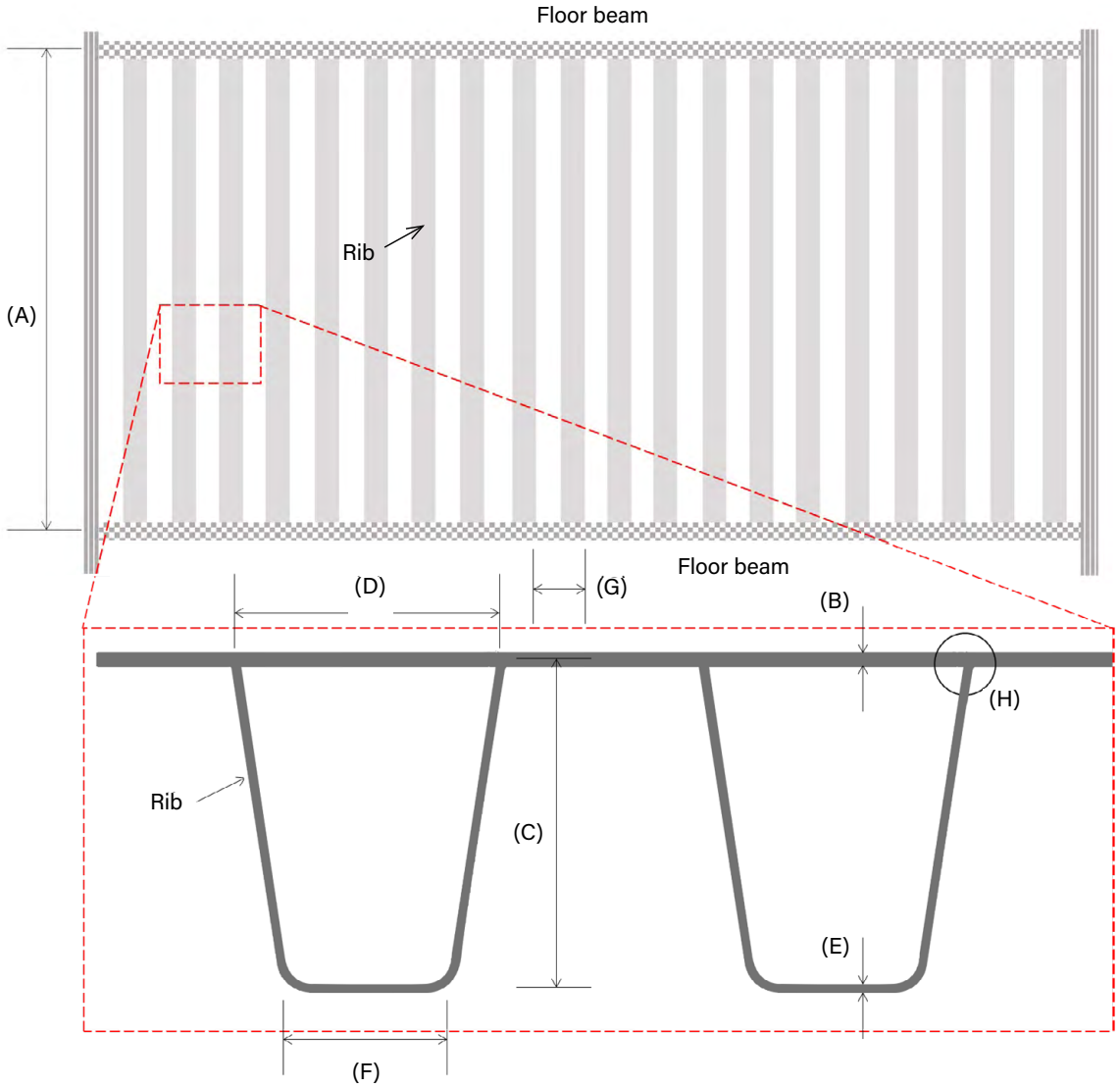
1. Strength design;
2. Serviceability (deflection) verification;
3. Verification of constructability; and
4. Fatigue verification.

For the fatigue verification, the three design levels defined by AASHTO [3] are reviewed in [1], and situations warranting their application are identified as follows:

- Level 1 is performance verification by testing;
- Level 2 is simplified 1D or 2D analysis; and
- Level 3 is 3D analysis.

There are situations where each design level may be appropriate, depending on the design check being performed and the availability of test data. In Appendix 1, a simplified version of the design flow chart in [1] is presented, with dimensioning guidelines suggested as a preliminary step.

Figure 1: Orthotropic deck schematic with key dimensions labelled. Spacing of floor beams (A); deck in [1] thickness (B); rib depth (C); upper rib width (D); rib thickness (E); lower rib width (F); centre-to-centre rib spacing (G); PJP welds between ribs and deck plate (H).



2.3 Wearing Surface Performances

The life of an OSD wearing surface can be five or fewer years if careful measures are not taken in the deck design and wearing surface application. Most infrastructure owners would find this an unacceptably short service life. The adequacy of the wearing surface is a function of its bond strength, ductility, fatigue strength, durability, waterproofing, and ability to resist chemicals such as de-icing salts [4]. The typical dimensions of a wearing surface vary between 25 and 50 mm because it must be thin enough to capitalize on the lightweight properties of OSDs, while being thick enough to cover any geometric irregularities on the deck plate surface [4]. Wearing surface performance depends on the bind strength, deflection of the deck, environmental exposure conditions, wearing surface thickness, material properties (e.g., abrasion resistance), and maintenance [4]. Excessive deflection introduces repeated shear stresses along the bond line of the wearing surface and tensile fatigue stresses in the wearing surface region on top of the ribs [4]. Conventional wearing surfaces have been reported to fail prematurely in cold regions that experience severe temperature fluctuations, as exemplified by the Yukon River Bridge in Alaska, due to the additional thermal stresses [5]. It is suggested in [5] that the fatigue performance of the wearing surface can be optimized by increasing the modulus ratio between the wearing surface and steel, increasing the deck plate thickness, and reducing the spacing between the floor beams and ribs.

The most common types of wearing surfaces used in North America are [6]:

- Bituminous surfacing systems;
- Concrete surfacing systems; and
- Polymer surfacing systems.

Different wearing surface types experience different stresses based on their thermal properties and effect on the deck stiffness, which influence longevity. Generally, using systems that are less sensitive to temperature change and increasing the deck stiffness reduces susceptibility to cracking. Bituminous surfaces have been reported to perform well on relatively rigid OSD systems. However, their temperature sensitivity means they tend to soften at high temperatures and harden at low temperatures, making them especially susceptible to tensile cracking and rutting [6]. Concrete surfaces are heavier than bituminous ones, but they have less risk of debonding if shear studs are used, and can add to the deck stiffness, thereby reducing the overall live load stresses developed in the deck [6]. Polymer surfaces are a lighter and thinner alternative, but they have no influence on the deck stiffness. Delamination of the wearing surface from the deck is the most frequently reported performance issue in polymer surfaces [6].

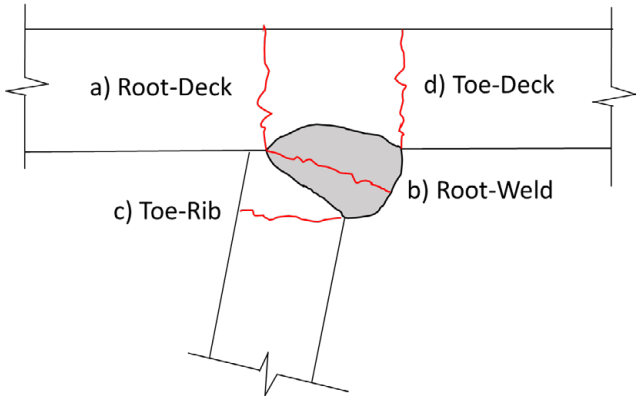
2.4 Fatigue Performance of OSD Welded Rib-to-Deck Joints

The welded rib-to-deck joints in OSDs are susceptible to fatigue cracking. Four types of fatigue crack can occur in these joints, including [7]–[10]:

- A weld root crack that propagates through the deck plate (root-deck crack);
- A weld root crack that propagates through the weld throat (root-weld crack);
- A crack that propagates from the weld toe on the rib through the rib wall (toe-rib crack); and
- A crack that propagates from the weld toe on the deck plate through the deck plate (toe-deck crack).

All four of these typical crack types are shown in Figure 2.

Figure 2: Illustration of different weld crack types.



Fatigue testing of OSDs is generally done in one of three types of experiments:

1. Single weld test specimen;
2. Single closed or U-rib test specimen; or
3. Large-scale test specimen containing several ribs.

The choice of experiment type influences the mode of fatigue failure observed. The prominent failure mode for single weld test specimens is cracking in the root-deck (Type *a*) [8]–[10], while fatigue tests on single U-ribs have been reported to experience weld toe cracking that propagates through the deck plate (Type *d*) [7], [14], [15]. Large-scale testing of OSDs has yielded both types of fatigue cracking depending on the loading method [16]–[18]. Single weld tests are suitable for assessing the impact of the welding process on fatigue behaviour, but lack the ability to replicate OSD structural system behaviour. Single U-rib experiments capture the integral behaviour of OSDs, but the prominently reported mode of failure (toe cracking) deviates from the more commonly observed root cracking that has been reported to occur in actual bridges [7], [20]. Large-scale testing provides the most accurate representation of the fatigue behaviour expected to occur in an actual bridge, at the price of complexity and cost of fabrication. The majority of large-scale tests [7], [16], [18] have reported higher propensity for root cracking. However, certain experiments have yielded toe-deck (Type *d*) cracking [17]. Large-scale tests also have the advantage of creating the actual redundant load path conditions present in OSD systems. Thus, as softening due to cracking at a rib occurs, large-scale tests have some ability to slow crack growth through redistribution of stresses, which will not be observed in a small-scale test.

Table 1 lists the three experiment types, the most common type of fatigue failure they observe, and their suitability for evaluating welded rib-to-deck joints in OSDs.

Factors that influence the fatigue performance of OSD rib-to-deck joints include:

- Weld geometry;
- Deck geometry;
- Wheel load transverse location; and
- Weld quality.

Research on the influence of each of these factors is briefly summarized in Sections 2.4.1 through 2.4.5.

Table 1: Fatigue Test Types for OSD Rib-to-Deck Welds

Fatigue Test Method	Most Common Failure Mode Observed	Suitability
Single weld test specimens	Root-deck (Type *a*) [8]–[10]	Suitable for assessing the impact of the welding process on fatigue behaviour but lack ability to replicate OSD structural system behaviour
Single U-rib test specimen	Toe-deck (Type *d*) [7], [14], [15]	Captures the integral behaviour of OSDs but mode of failure deviates from root cracking more commonly observed in actual bridges
Large-scale test specimen	Root-deck (Type *a*) and Toe-deck (Type *d*) depending on loading method [16]–[18]	Provides most accurate representation of fatigue behaviour in actual bridges, but much more complex and costly

2.4.1 Weld Geometry

The PJP weld used in the rib-to-deck joint is the most influential parameter on the fatigue performance of OSDs, with improvement in fatigue performance generally reported as the level of weld penetration (LOP) increases (e.g., [1,] [7], [11], [13], [14], [16], [17], [19]). The proportional improvement in fatigue performance with increasing LOP is only seen up to 75–80% penetration, after which the fatigue life of an OSD will remain the same or decrease slightly. Furthermore, increasing the LOP beyond 80% increases the risk of weld defects during fabrication, such as weld melt-through and blow-through, both of which can severely degrade the fatigue resistance of the welded connection.

Other aspects of weld geometry can also affect the fatigue life of OSDs, such as the weld profile [20] and the presence of hot cracks [21]. Hot cracks are shrinkage cracks that occur during the welding process when the liquid weld metal is insufficient to fill the voids between the solidifying weld metal. These types of cracks were reported to be the primary cause of extensive fatigue cracking in 66 of 408 deck panels in the Bronx-Whitestone Bridge [21]. Hot cracks reduce the residual compressive stresses in the weld, which increases the likelihood of fatigue failure [21]. Upon early discovery, hot cracks can be easily repaired, highlighting the importance of a thorough inspection for this defect. The selection of an appropriate filler metal and welding procedure can also reduce the risk of hot cracking.

One study compared three different weld profiles using numerical analysis and suggested that the root size should be strictly controlled [20]. A traditional single-sided weld, an oversized weld, and double-sided weld were examined, and the study concluded that the performance of the double-sided weld (which would be harder to implement practically in OSD fabrication) was significantly better than the traditional weld. The authors also demonstrated that increasing the inner weld pass size distributes the stresses in a manner that is beneficial to the fatigue life and shifts fatigue failure to the weld toe rather than the root. The weld profile comparison also showed that the traditional weld performed better than the oversized weld. It should be noted that the study did not investigate the effects of LOP.

2.4.2 Deck Geometry

Deck geometry helps to distribute the stresses from the wheel loads to the welds, which significantly influences OSD fatigue behaviour. Increasing deck plate thickness is the most effective way to improve fatigue performance, because the most commonly reported forms of fatigue damage are cracks propagating from either the weld toe or root through the deck plate [9], [20]. Increasing the deck plate thickness to 16 mm can improve fatigue performance. In contrast, larger rib thickness has a limited benefit on fatigue performance [9], [22], but using larger overall ribs can be beneficial [9]. One study also showed that pre-cambering of decks improved fatigue performance [23].

2.4.3 Transverse Loading Location

Two numerical studies investigated the effect of the transverse load location on the stresses surrounding the rib-deck joint weld [14], [17]. One evaluated the nature of the stresses at the ribs, membrane stresses, and deck plate stresses surrounding the weld. The load was placed at thirty varying transverse locations between the weld of interest and the two adjacent ribs. The results showed that the deck plate and membrane stresses were maximized when the load was on top of the weld, while stresses in the ribs were maximized when the load was on the adjacent rib [14]. The second study investigated three scenarios: load centred on the rib wall away from the analyzed weld (Case 1), load centred on the rib wall on top of the joint (Case 2), and load centred on the rib (Case 3). The results showed that Cases 1 and 2 had the highest likelihood of developing cracks that propagated into the deck plate, while Case 3 had a higher likelihood of developing a root-weld crack. The highest overall stress in the weld resulting from each case was Case 1, followed by Cases 2 and 3 [17].

2.4.4 Fatigue Analysis Methods

Finite element (FE) analysis is a powerful tool that can be leveraged to provide fatigue life predictions using various techniques, each with its respective advantages and limitations. The fatigue performance prediction of OSDs can be performed with the following approaches:

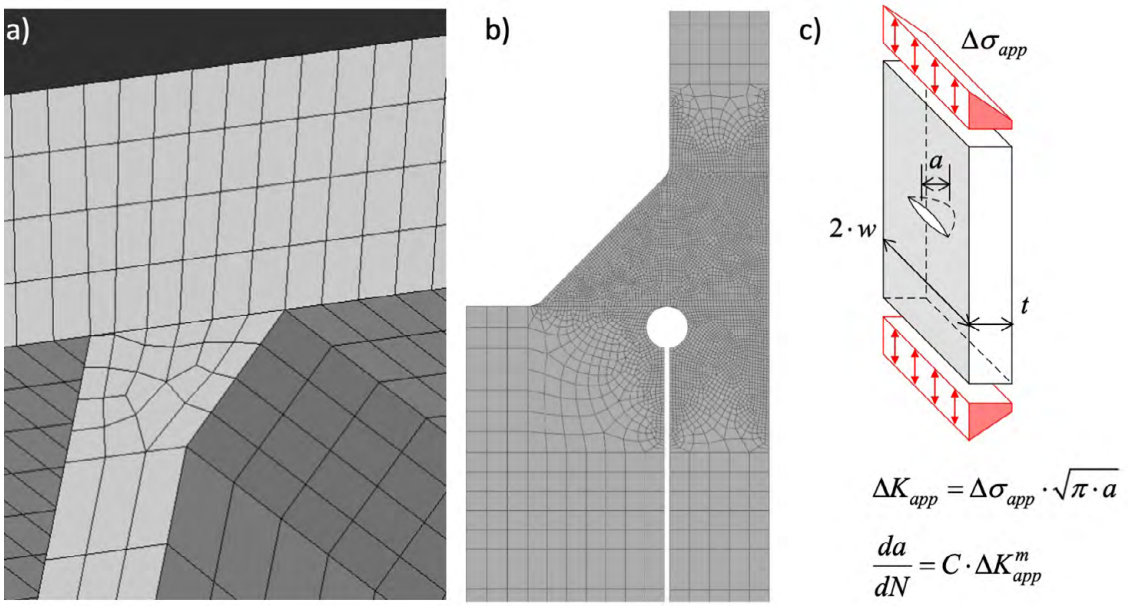
1. Hot-spot stress approach;
2. Notch stress approach; and
3. Linear elastic fracture mechanics (LEFM).

All three approaches are illustrated in Figure 3.

The hot-spot and notch stress approaches are used to determine characteristic local stresses at or near the critical fatigue location, and then use established empirical curves from tests to determine the fatigue life based on the calculated stress range (referred to as S-N or "stress-life" curves). The LEFM approach does not require empirical curves, as the fatigue life is directly calculated by evaluating the crack propagation rate through fracture mechanics [24].

The hot-spot stress method for the fatigue assessment of welds is discussed in several international design codes and guidelines, including those from the International Institute of Welding (IIW) [24], the European Committee for Standardization (CEN) [25], and the Japanese Society of Steel Construction. Traditionally, the hot-spot stress method has been used to evaluate the fatigue life of weld details for which the toe is the critical crack location. Research has shown that the notch stress approach is also a viable method for analyzing fillet welds of OSD rib-deck joints, often yielding conservative fatigue life estimates compared to the hot-spot stress method when analyzing the weld toe. LEFM is a versatile fatigue life prediction tool, but it is greatly influenced by the assumed crack properties, including direction, size, and propagation mode. This means that LEFM requires more expertise to implement and yields results that can be highly sensitive to assumptions regarding the input parameters. LEFM is also limited in that it only considers the crack propagation phase of the total fatigue life. However, previous studies have shown that for the assessment of welded structures, for which small initial defects due to fabrication are unavoidable, this is not a significant issue.

Figure 3: Modelling features of three fatigue design methods: a) hot-spot stress, b) notch stress, c) linear elastic fracture mechanics (LEFM).



2.4.4.1 Hot-Spot Stress Method

In the context of FE analysis, the weld stress of interest for fatigue design typically occurs at the weld root or weld toe, which are edge regions that are mesh sensitive. This means that the stress at those points will tend to infinity the smaller the mesh becomes, which presents a challenge in obtaining a characteristic design stress. The hot-spot stress method overcomes this challenge by extrapolating the stress from reference points near the region of interest to obtain the nominal stress. The method is typically used to evaluate the structural stress associated with a fillet or full penetration weld. Studies have shown that this technique becomes less accurate when used to predict the stress at the weld root [1], [10], [20], which is the most reported form of fatigue damage occurring in actual OSDs in bridges. The reasons for this are that the weld root often represents a sharper change in geometry than the weld toe, and if there is any lack of weld penetration, the weld root can have a significant pre-existing crack-like defect present at the end of the fabrication process (which can vary greatly in size), before fatigue loading even starts. Note that the hot-spot stress method is not limited to FE-based applications and can also be used experimentally to predict the stress at the weld of an OSD by extrapolating the measured strains from nearby reference points [13]. AASHTO recommends this method for fatigue analysis of the weld toe of the rib-deck joints of OSDs in the absence of experimental data for a similar geometry of the OSD of interest [3].

2.4.4.2 Notch Stress Approach

The notch stress approach provides an alternative solution to obtaining the weld stress at a mesh-sensitive region of interest. It is more versatile than the hot-spot stress method in that its application has not been limited to weld toe-like fatigue details. Researchers have used the notch stress approach to analyze the stresses at both the weld root and the toe of OSD rib-deck joints [17], [26]. An effective notch root radius of 1 mm has yielded consistent results. The notch stress approach requires a very fine mesh, which makes modelling entire deck panels prohibitively costly from a computation time perspective, unless creative partial models or sub-modelling strategies are employed.

2.4.5 Linear Elastic Fracture Mechanics

The LEFM approach allows fatigue life to be predicted by calculating the crack propagation rate per loading cycle through the Paris-Erdogan crack growth law. This method requires knowledge of the crack growth constants C and *m*, which can be obtained through the literature or through testing. Crack growth is proportional to an applied stress intensity factor (SIF) range, ΔK_{app}, which is a function of the crack size, stress range, and detail geometry. Theoretically, the LEFM approach can be applied without FE analysis, but it is often coupled with FE analysis to analyze complex geometries, such as those present in OSDs. The SIF range calculation is highly dependent on the mode of cracking, which can be one of three types: Mode I: opening and closure of cracks, Mode II: in-plane shear, or Mode III: out-of-plane shear. For practical purposes, Mode I loading is often assumed to dominate and phase effects are conservatively ignored. This approach has been used to understand propagation behaviour and provide practical fatigue life predictions for OSDs for root-deck plate cracks (Type *a*) [27], as well as toe-deck plate cracks (Type *d*) [28] and root-weld cracks (Type *b*) [29] (see Figure 2).

3 Survey of Professional Experience

3.1 Background

A survey of experts from various industry sectors was conducted to better understand the challenges associated with OSD design and to better understand the barriers preventing increased use of OSDs on a broader range of project types.

The survey was distributed to the Project Advisory Panel, members of the CSA S6 Section 10 Technical Subcommittee, and members and contacts provided by the Canadian Institute of Steel Construction. The exact number of people who received the survey could not be accurately tracked, given the “word of mouth” distribution approach, which was adopted to maximize the number of responses. In total, five individuals with experience working with OSDs completed the survey. Of these, four participants were bridge designers/consultants working in Canada, and one was a

Canadian structural steel fabricator with experience in OSD fabrication for recent projects. Regarding experience with OSDs in Canada, most participants only had experience with rehabilitation projects.

3.2 Survey Questions

The survey consisted of the following questions:

Q1: What were the common deciding factors for using OSDs for projects in which you were involved?

Q2: Please describe any factors you are aware of contributing to OSDs not being the preferred bridge decking alternative for other past projects.

Q3: If applicable, please describe any challenges faced using the existing OSD design provisions.

Q4: Please provide feedback on how the existing OSD design provisions, and in particular those in the CSA S6 code, could be improved.

Q5: Please describe specific aspects of the OSD design process in which you would have benefited from expanded design guidance and/or design examples (if applicable).

Q6: Please list the OSD projects you have worked on within Canada.

Q7: Please list the OSD projects you have worked on outside of Canada.

3.3 Survey Responses

Survey responses were collected through both written replies and through interviews. Participants' feedback for each question is summarized in the following paragraphs.

Q1: What were the common deciding factors for using OSDs for projects in which you were involved?

- Globally, OSDs have only been considered on very long-span bridges, where the weight of the deck is important and can have a significant impact on the cost of construction and the weight of the superstructure elements. Otherwise, they are also sometimes used for bascule bridges, where again the weight saved is worth the increase in fabrication cost.
- OSDs result in reduced weight to reduce towers and foundations.
- For a new bridge, it is often only advisable to use an OSD if there is no alternative. The long-term maintenance issues with paving and fatigue are too great. For rehabilitation, it often makes more sense due to the weight saving (i.e., it is worth taking a risk that there will be long-term maintenance issues to avoid the cost of replacing the bridge).
- An OSD was used for an existing bridge that required a deck replacement and widening to meet functional roadway needs. The existing rolled beam girders were in good shape and were kept. The OSD was a lightweight system that allowed the deck to be widened with the existing rolled beams as desired — this would not have been possible with a heavier deck.

Q2: Please describe any factors you are aware of contributing to OSDs not being the preferred bridge decking alternative for other past projects.

- OSDs are very expensive to fabricate (compared to concrete decks). Few fabricators in North America can do it (many will claim they know how, but the actual experience may be low, and it is easy to get wrong).
- Surfacing on OSDs is often an issue. Additional asphalt cannot simply be added (as with many concrete bridge decks), and the additional weight of thick surfacing offsets the low weight of the OSD. An example is Lions Gate Bridge, which has many surfacing issues.

- OSDs are expensive compared with a reinforced or prestressed concrete deck. Fatigue life is poor unless you have a thick deck plate (16 to 20 mm depending on traffic), transport of deck pieces is expensive, and site welding of splices can be unreliable, so contentious and costly to correct.
- The long-term issues are fatigue and paving issues (paving on the decks is very problematic to replace and often fails prematurely, causing all sorts of issues for the owner and users).
- One respondent indicated they are sometimes consulted in the preliminary phase of design for cost estimates. Cost is a common obstacle. There is a lack of experience and confidence from the designers when preliminary options are discussed.
- Concerns over fatigue performance, maintenance demands, capital cost, and lack of comfort with the system are primary issues.

Q3: If applicable, please describe any challenges faced using the existing OSD design provisions.

- One recipient stated, "I've only looked at rehab on North American OSDs, where the specific provisions in AASHTO are either not applicable or require analysis from first principles. Therefore, I can't really comment on this item."
- The tolerance on the rib-to-deck welds can be a challenge. The typical requirement is 80% penetration with a +/- range, but with a typical 8 mm thick rib, the +/- can be difficult to achieve.
- The CSA provisions rely heavily on AASHTO.

Q4: Please provide feedback on how the existing OSD design provisions, and in particular those in the CSA S6 code, could be improved.

- CSA S6 could draft its own provisions (not just reference AASHTO).
- AASHTO provisions are confusingly split into Sections 6.14.3 and 9.8.3.
- Additional clear diagram explaining the design flow would be helpful.
- Europe and Japan have good codes for OSDs. It would be worth consulting them. In general, the Eurocode appears to treat stiffeners/stiffened panels in a more general way. Not all OSDs use troughs; some use plate stiffeners, which shouldn't be any different from a stiffened plate element (see EN 1993-1-5 Section 4.4 and 4.5).
- OSDs are tricky and should only be used by knowledgeable designers and owners. The weld penetration levels and QA/QC on the welds are critical. Furthermore, additional guidance/clarity around paving would be beneficial.
- CSA S6 could introduce more consistent details (e.g., standard rib profile, spacing, etc.).

Q5: Please describe specific aspects of the OSD design process in which you would have benefited from expanded design guidance and/or design examples (if applicable).

- EN 1993-1-5 gives rules for stiffened elements and treats them more generically. This makes more sense as a code provision, but possibly could cause confusion for other parts of CSA S6 Section 10 (e.g., box girders, longitudinally stiffened elements, etc.).
- The diagrams in the Eurocode for buckling of a stiffened panel are useful.
- It would be helpful to guide designers in choosing cut-outs below troughs (like the Eurocode provisions) or fully welded round bottom troughs passing through cut-outs. This depends on diaphragm/floor beam stiffness.
- It would be helpful to do a fatigue comparison of Japanese-type diaphragm spacing (about 2 m or so even with closed stiffeners) with the more common Canadian spacing of about 4 m.

Q6: Please list the OSD projects you have worked on within Canada:

- A list of projects participants reported working on in Canada is provided in Table 2.

Table 2: OSD Projects Within Canada that Surveyed Individuals Worked On

Projects Worked on Within Canada	
Project	**Location**
Lions Gate Bridge	Vancouver, British Columbia
MacKay Bridge	Halifax, Nova Scotia
Macdonald Bridge	Halifax, Nova Scotia
Saint John Harbour Bridge	Saint John, New Brunswick
Tender design for new Champlain Bridge	Montreal, Quebec
Proposal design for flyover replacement on West Georgia Street (Georgia Viaduct)	Vancouver, British Columbia
Red Willow River Bridge	near Beaverlodge, Alberta

Table 3: OSD Projects Outside Canada that Surveyed Individuals Worked On

Projects Worked on Outside Canada	
Project	**Location**
Queen Elizabeth II Bridge	England
Erskine Bridge	Scotland
Severn Bridge	Wales
West Gate Bridge	Australia
Osman Gazi Bridge	Turkey
1915 Çanakkale Bridge	Turkey
First and Second Bosphorus Bridges	Turkey
Bascule bridge	Saudi Arabia
Humber Bridge (only doing repairs)	England

Q7: Please list the OSD projects you have worked on outside of Canada:

- A list of projects participants reported working on outside of Canada is provided in Table 3.

Other topics discussed by survey participants:

- Rehabilitation: OSDs are commonly used for existing bridge rehabilitation/widening projects and when construction time/traffic disruption is a key consideration.
- New construction: OSDs are typically only the preferred deck alternative for very long-span suspension bridges (i.e., self-weight critical and foundation costs) and moveable bridges (e.g., lift, bascule, and swing bridges). One participant shared reflections on the prospects for developing OSD systems for medium-span girder bridges. Given the durability problems encountered with concrete decks, the potential benefits are intriguing if the durability issues with OSDs could be solved. To make this work, designing for composite action between the deck and the girders would be important but would also add complexity to the design.

- Experienced fabricators recognize that constructability of an OSD has a bigger impact on the fabrication cost than reducing material. The idea of a "sweet spot" for the deck plate thickness to minimize fabrication cost was shared in several discussions.
- An average score of 3.5 out of 5 was obtained when participants were asked to rate the clarity of the current OSD design provisions in CSA S6.

Overall, the challenges and considerations participants encountered working with OSDs included:

- The relatively high capital cost due to fabrication of the decks is an issue.
- The current provisions do not distinguish between local and global compression members.
- Identifying the appropriate deck plate thickness during the design stage is critical.
- Adhering to the differential deflection limits between adjacent ribs can be a challenge.
- After operation, longitudinal cracking in the wearing surface required frequent repairs.
- Alignment between panels along transverse welded joints can present a challenge.
- Moisture entering ends of ribs (splices) can be an issue.
- Bridge geometry can sometimes require odd spacing of the ribs and/or the addition of open ribs at some locations.
- Fabricators can struggle with PJP welds (i.e., achieving desired level of penetration, avoiding blow-through). The current penetration that is required is not based on research, and adequate or consistent inspection of partial penetration welds can be a challenge.

3.4 Survey Response Summary

The survey and the expert discussions it stimulated provided valuable information about challenges associated with OSD design and barriers to increased OSD use on a broader range of project types.

Participant comments suggest that some immediate modifications to the existing CSA S6 OSD design provisions, which reference many different AASHTO clauses, could provide improved clarity for designers. This could be accomplished by providing, for example, a design flow chart that lists all the relevant AASHTO clauses and suggests when they should be referenced. Alternatively, all of the necessary clause language could be coherently added to CSA S6 instead of referencing AASHTO.

Two additional topics that participants frequently raised were fatigue performance and wearing surface durability, which are a major concern for bridge designers and owners, and more guidance on design and best practices could be included in CSA S6. Finally, participants noted that OSDs are an attractive alternative for bridges where weight is critical (e.g., very long-span bridges and moveable bridges), but the potential long-term durability issues from fatigue and wearing of the pavement, combined with relatively high capital costs, may suggest that OSDs should not be used where the benefits do not outweigh the limitations. The high capital costs come largely from the cost of fabrication labour.

4 Structural Analysis Methods

In this section, the methods used to implement the current design procedures in CSA S6:19 (referencing AASHTO where appropriate) are described for the deflection and fatigue design of a typical OSD system. Results of the analysis are presented in Section 5.

4.1 Deflection Analysis

According to CSA S6:19, OSD designs must meet the AASHTO requirements for deflection. However, the CSA S6 and AASHTO design trucks differ in both total weight and axle distribution, as shown in Figure 4. The deflection limits restrict the deck deflection and the extreme relative deflection between adjacent ribs. Specifically, AASHTO Clause 2.5.2.6 limits deck plate deflection due to vehicle load to span/300, rib deflection due to vehicle load to span/1000, and inter-rib deflection under vehicle load to 0.1 in (2.54 mm).

Figure 4a: CSA S6 design truck dimensions and loads [2].

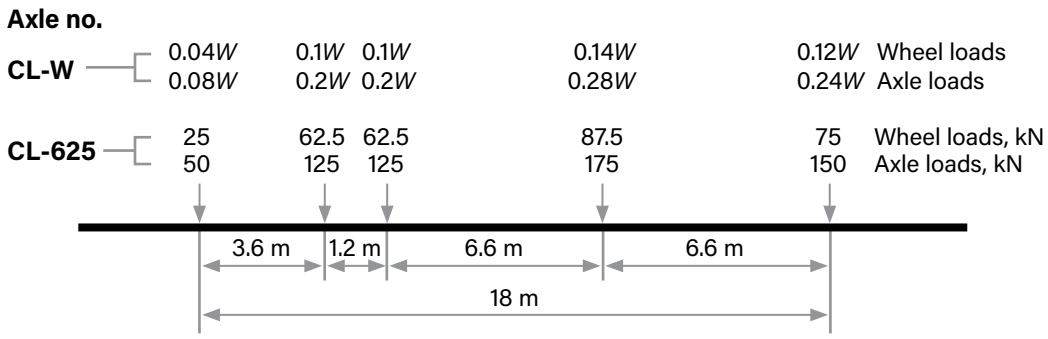
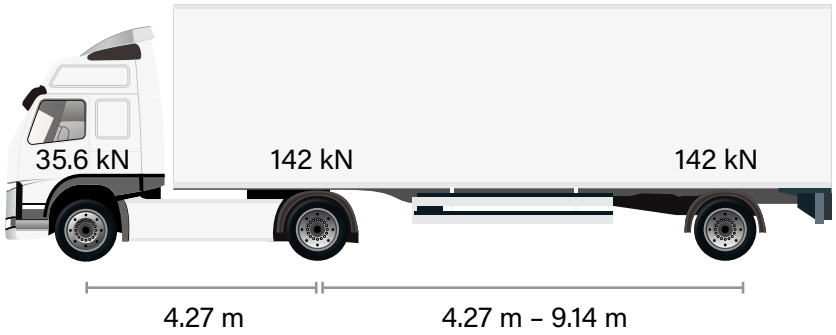


Figure 4b: AASHTO design truck dimensions and loads.



SAP2000 v17 software was used to perform a simple longitudinal (2D) analysis to determine axle locations that would result in maximum deflections for both the CSA and AASHTO design trucks. In general, a 2D analysis is insufficient to capture peak deflections due to the two-way behaviour of the decks and the assessment requirement for the relative deflection between the ribs. Therefore, a 3D FE model of an existing deck was also built using Abaqus 2018 software. Critical dimensions are provided in Figure 5. Analyses for different deck spans were also conducted using the 2D model to compare the deflection resulting from each truck over a wide range of possible deck spans.

It is believed that a primary objective of the AASHTO defection limits is to ensure adequate durability of the wearing surface. One question raised by experts during the industry survey is whether the current provisions unnecessarily penalize Canadian bridges, given that they require the AASHTO deflection limits be met with the heavier Canadian design truck. An important objective of this analysis was to quantify the significance of this penalty as a starting point for further discussion.

Figure 5: Cross-sectional dimensions of modelled deck.

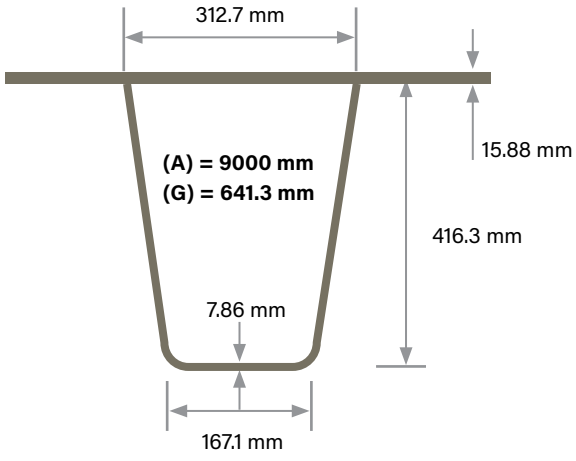
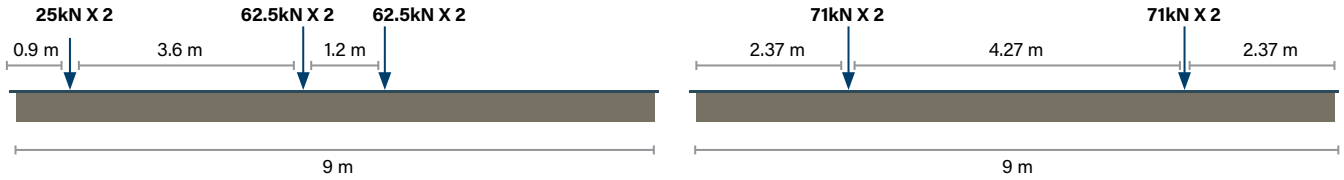


Figure 6: Side view of deck showing location of loads that maximizes deflection for Canadian design truck (left) and American design truck (right).



4.2 2D Influence Lines Analysis

For the 2D influence line analysis, frame elements were modelled in SAP2000 with the same moment of inertia (MOI) as the deck. The SAP2000 section designer option was used to obtain the MOI.

Simple support conditions were modelled, and nominal steel properties were assumed including a modulus of elasticity of E = 200 GPa, which is the value specified for steel in CSA S6:19 [2]. The load locations that resulted in maximum deflection for both trucks are shown in Figure 6.

4.3 3D FE Analysis

The modelling procedure used in this study adhered to the provisions in AASHTO Clause 4.6.3.2 [3] for a refined 3D analysis, which includes assuming:

- Linear elastic material properties;
- Small deflection theory;
- Plane sections remain plane;
- Neglecting residual stresses and imperfections; and
- Idealizing weld geometry (however welds were neglected during the deflection analysis).

Figure 7a: Top view showing entire deck and reduced geometry model, illustrating the location of Canadian wheel loads that maximize deflections.

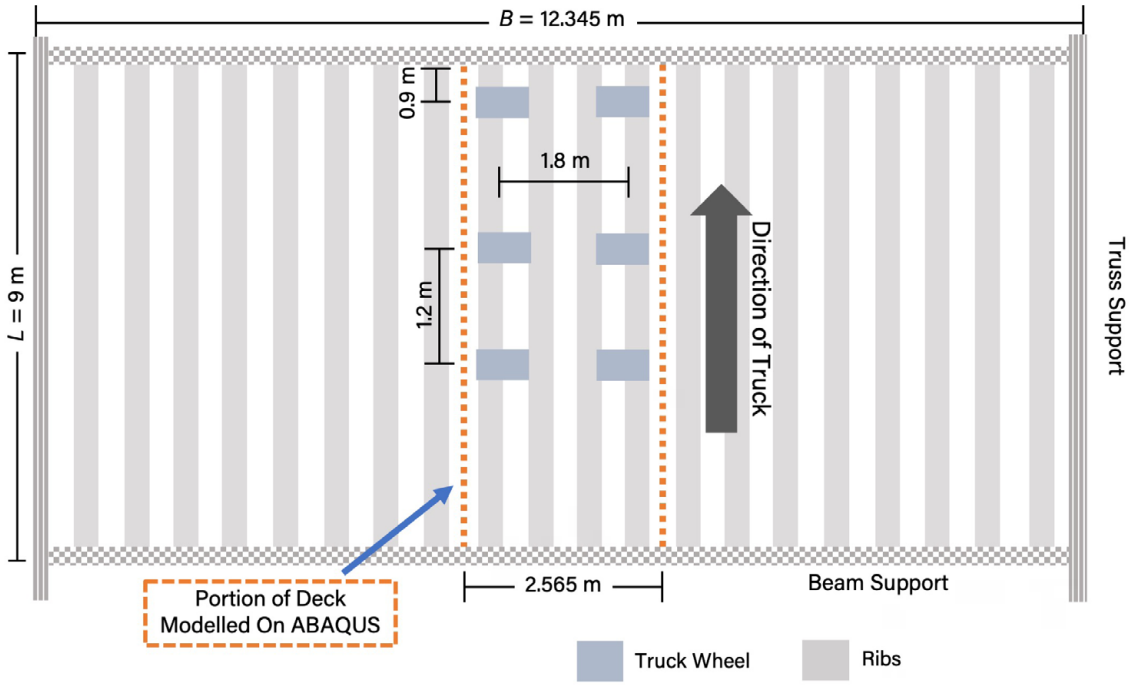
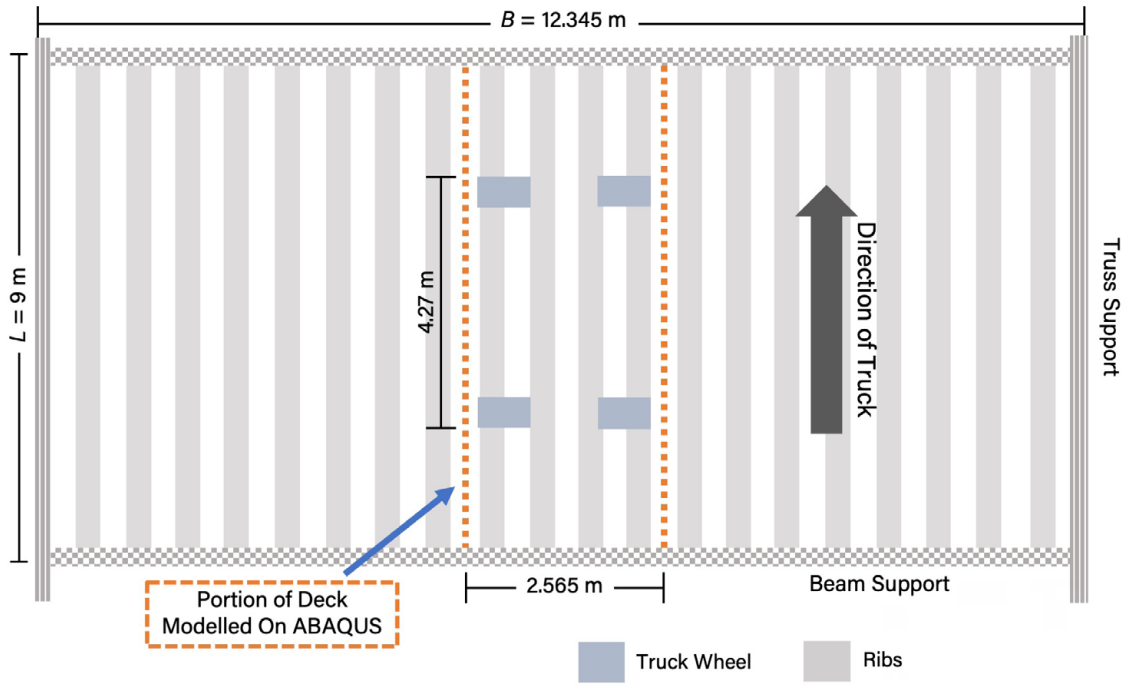


Figure 7b: Top view showing entire deck and reduced geometry model, illustrating the location of American wheel loads that maximize deflections.



A first order elastic analysis was performed. The steel was again assumed to have a modulus of elasticity of E = 200 GPa, with no yielding assumed. A pressure load was applied with a design tire contact area of 254 mm x 508 mm for the American truck and 250 mm x 600 mm for the Canadian truck as prescribed in the respective standards. The load locations were obtained from the 2D influence lines analysis.

Per AASHTO, the mesh specification for the deflection analysis requires a sufficiently refined mesh to resolve pressure loading with reasonable accuracy. This was accomplished by applying a mesh size of t_{deck} / 4 for the deck, while using a coarser mesh of 2 · t_{rib} for the ribs (where t = thickness). A 3D solid continuum linear 8-node brick element with reduced integration (C3D8R) was used.

Only a portion of the deck was modelled for computational efficiency per AASHTO Clause 4.6.2.6, which allows the use of an effective flange width for decks with panel aspect ratio, L / B < 5 (note: L = 9 m and B = 12.345 m in this case, Figure 7). The effective flange width, be, was calculated to b_e 2.565 m. The resulting modelled portion of the deck is shown on a sketch of an entire panel in Figure 7.

The ends of the existing deck were simply supported, while the cut-out sides for the reduced geometry were restricted from lateral translation (U_x) and rotation (θ_z) about the longitudinal axis, as shown in Figures 8 and 9, where U refers to translation and θ refers to rotation, and the subscript denotes the relevant axis, as indicated in Figure 8 (Y = vertical, Z = longitudinal, and X = lateral). The restraint was applied to the deck plate between the ribs and the ribs were supported on the bottom at their ends. The maximum deflection and the extreme relative deflection between the ribs were recorded.

4.4 Varying Span Analysis

To study the effects of span on the comparison of deflections obtained using the CSA and AASHTO provisions, the 2D analysis was repeated with spans of 3 m, 9 m, and 27 m. The modelled spans were subjected to loads from the CSA and AASHTO design trucks to obtain the results for maximum deflection using a moving load analysis. While the applicability of the 27 m span analysis to OSDs could be questioned, the goal of this study was to model cases where the panel length is longer and shorter than the design truck lengths, to cover the full range of possible situations that could arise.

Figure 8: Isometric view of modelled deck with boundary conditions (BCs) indicated.

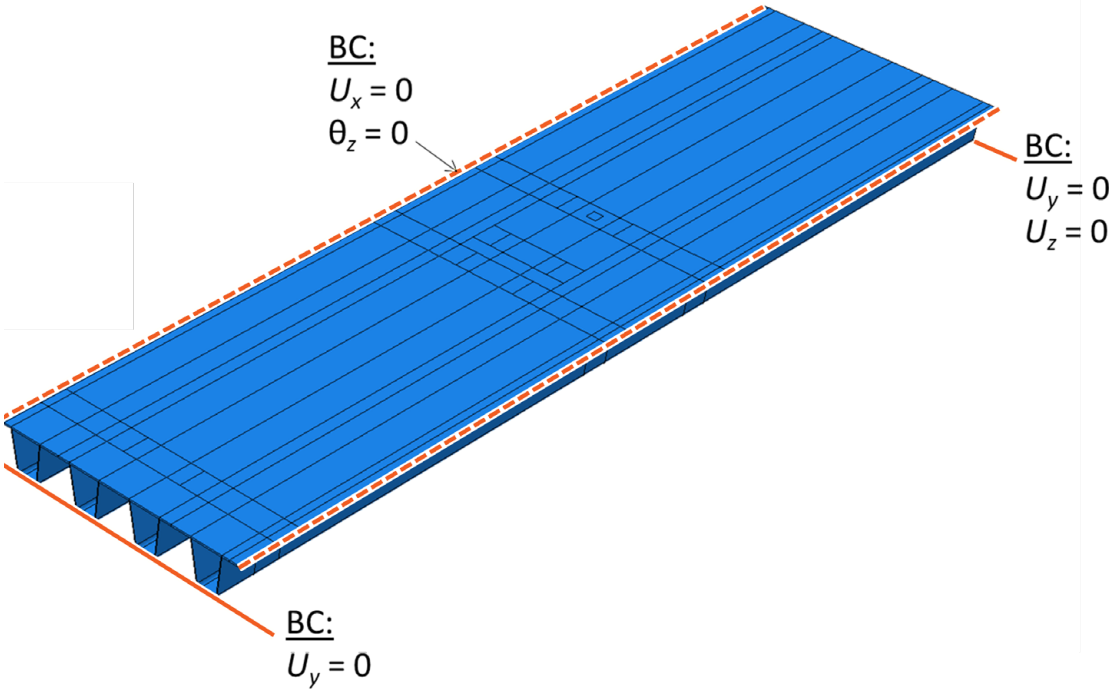


Figure 9: Defc

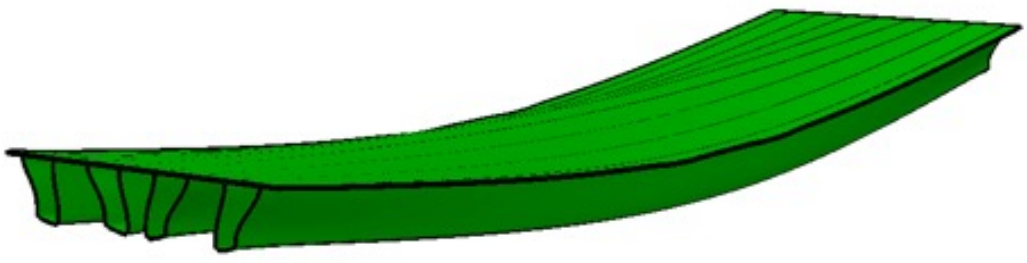


Figure 10: Span analysis load locations for CSA S6 (left) and AASHTO (right) trucks.

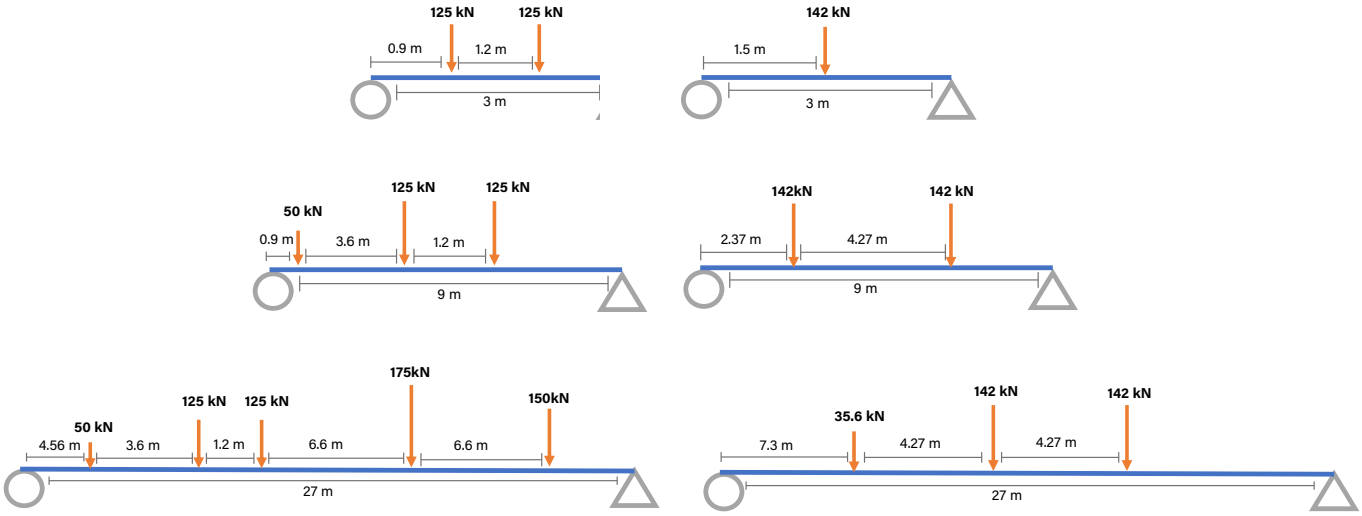


Figure 10 shows the load locations that result in the maximum longitudinal deflection for the different spans, with the Canadian truck on the left and the American truck on the right. In the figure, a roller support and a pin support are denoted with a circle and a triangle symbol, respectively.

4.5 Fatigue Analysis

The fatigue analysis was conducted on the same deck system as the one modelled in the deflection analysis, with a reduced geometry and rib-to-deck welds added. For this analysis, a finer mesh was used to determine local stresses in the weld region.

4.5.1 Hot-Spot Stress Analysis

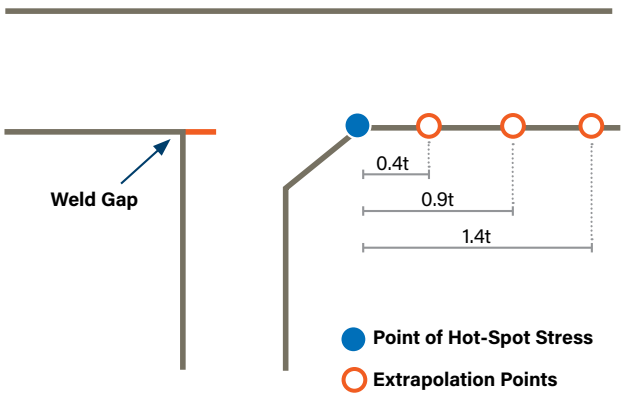
A refined analysis described in Clause 4.6.3.2 of AASHTO (Level 3 design) was used to determine the local stresses at the rib-deck plate connection weld. Refined analysis is generally required for fatigue design using the hot-spot stress method. The characteristic stress of interest typically occurs at the weld root or weld toe, which are edge regions that are mesh sensitive, meaning that the stress at these locations will tend to infinity as the mesh is further refined. The hot-spot stress method overcomes this challenge by extrapolating the stress from three points at distances $0.4 \cdot t$, $0.9 \cdot t$ and $1.4 \cdot t$ near the region of interest, where t is the deck plate thickness, as illustrated in Figure 11.

The following expression is used to calculate the hot-spot stress, $\sigma_{hs,}$ according to the IIW [24]:

$$\sigma_{hs} = 2.52 \cdot \sigma_{0.4 \cdot t} - 2.24 \cdot \sigma_{0.9 \cdot t} + 0.72 \cdot \sigma_{1.4 \cdot t}$$

where t is the deck plate thickness and $\sigma_{0.4t,}$ $\sigma_{0.9t,}$ and $\sigma_{1.4t}$ are the stresses at the reference points.

Figure 11: Illustration of hot-spot stress showing extrapolation points.

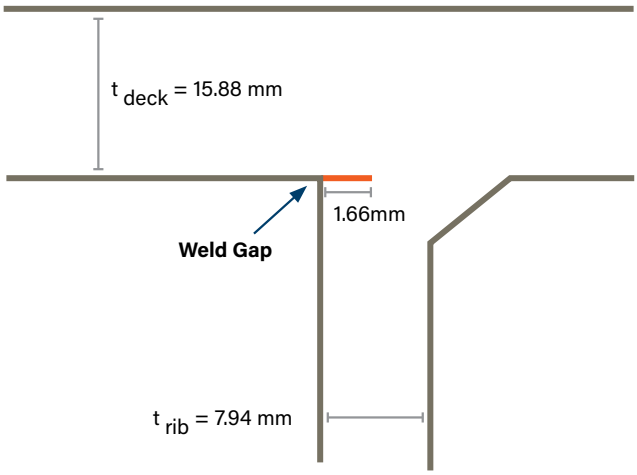


The same general modelling assumptions used in the deflection analysis were used in the fatigue analysis with the following exceptions:

- A finer mesh was used to evaluate local stresses at the welds;
- A "half model" of the deflection analysis was used to further leverage symmetry; and
- The application of wheel loads was based on the fatigue requirements of AASHTO and CSA S6:19.

The modelled weld was a PJP weld with 80% penetration, as shown in Figure 12.

Figure 12: Assumed dimensions of rib-to-deck weld. t_{deck} = deck thickness, t_{rib} = rib thickness.

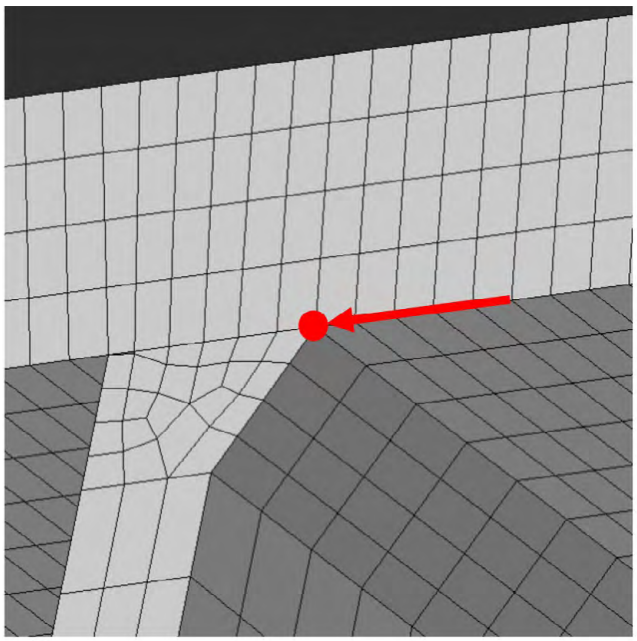


The element thickness in the rib was t_{rib} / 2, while the mesh size for the deck was t_{deck} / 4, as shown in Figure 13. The mesh size in the weld varied between t_{rib} / 2 and t_{deck} / 4. The mesh size in the weld region is as important as the mesh size in the extrapolation region where the hot-spot stress method is applied.

4.5.2 Loading Cases

Per CSA S6 Clause 10.16.4, the AASHTO-prescribed 508 mm x 254 mm tire contact area was used for both the American and Canadian cases. The AASHTO

Figure 13: Mesh of rib-to-deck weld with red dot representing weld toe at which stress is extrapolated to calculate hot-spot stress.



code prescribes a different American truck for fatigue analysis than the one used for deflection analysis, as shown in Figure 14. This is significant because the fatigue truck is about 30% longer and distributes the same weight to five axles instead of three. As noted, the CSA provisions require the use of the American tire contact area for fatigue analysis and instruct that users need only consider the local forces resulting from the tandem 125 kN axles.

The resulting model boundaries and tire footprints for the fatigue analysis are presented in Figure 15.

4.5.3 Boundary Conditions

Figures 16 through 18 detail the boundary conditions for the FE model used in the fatigue analysis, including the boundary conditions on the support line (Figure 16), as well as the longitudinal (Figure 17) and transverse (Figure 18) lines of symmetry. In these figures, the boundaries are indicated with dashed lines.

Figure 19 shows the deflected shape of the deck panel under the prescribed loading.

Figure 14: Dimensions of AASHTO fatigue limit state (FLS) truck for OSDs, and serviceability/ultimate limit state (SLS/ULS) truck.

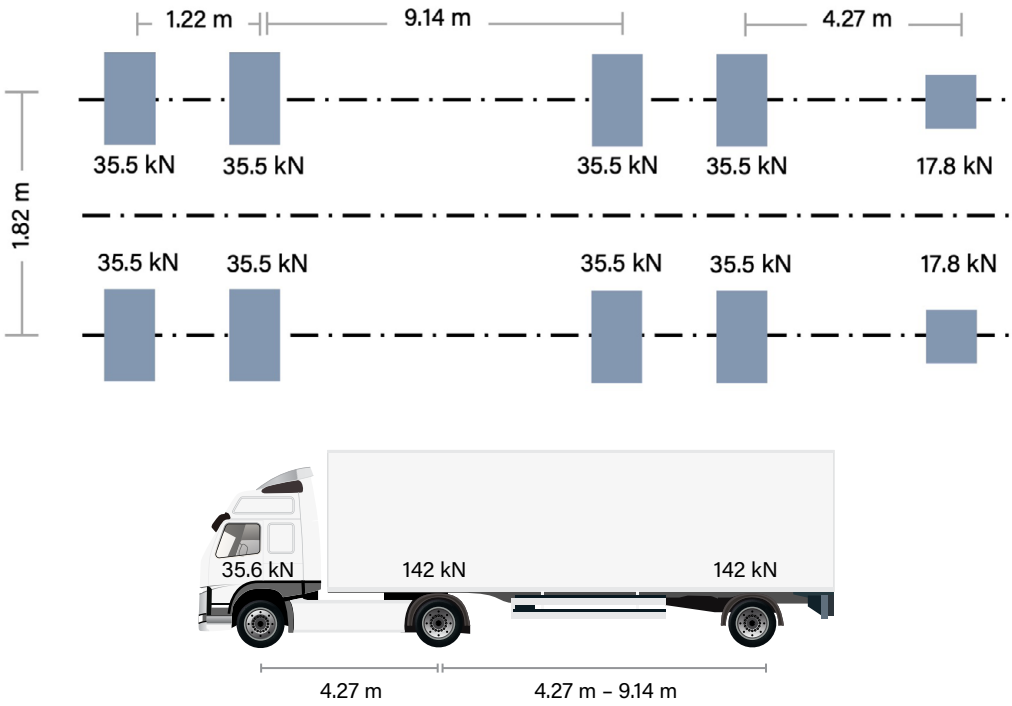


Figure 15: Top view of deck showing reduced geometry model, weld location, and truck wheel locations used in fatigue analysis.

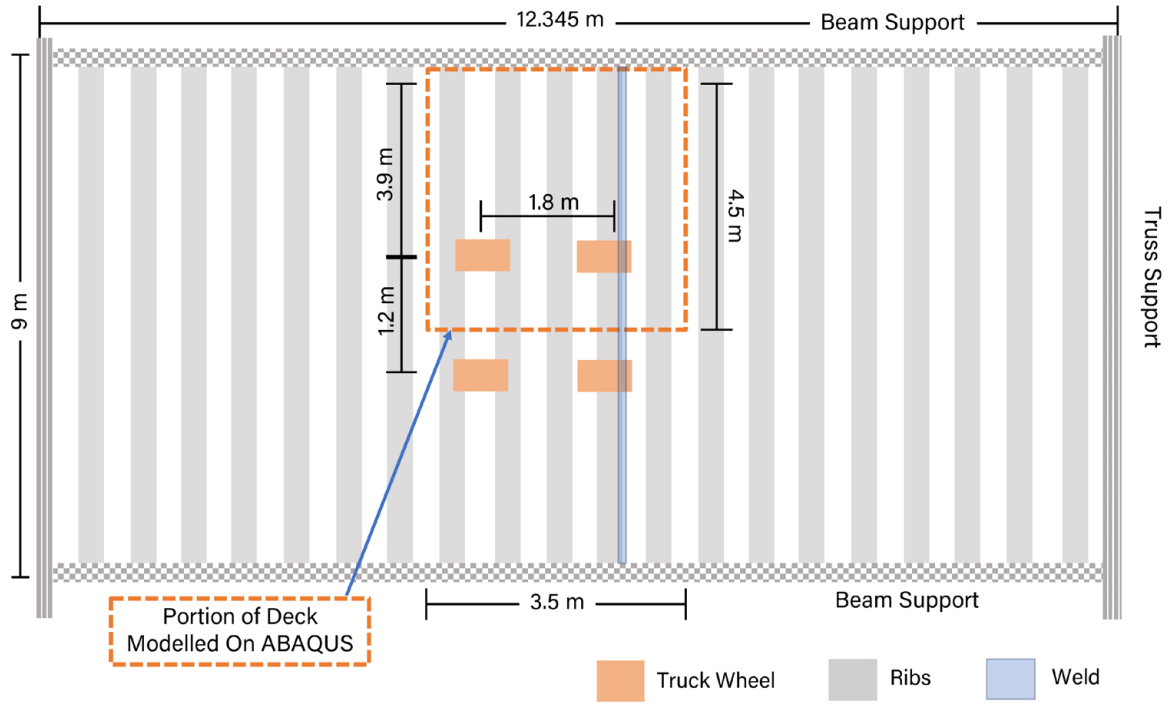


Figure 16: Isometric view of modelled deck — Boundary Condition (BC) 1: $U_x = U_y = U_z = 0$.

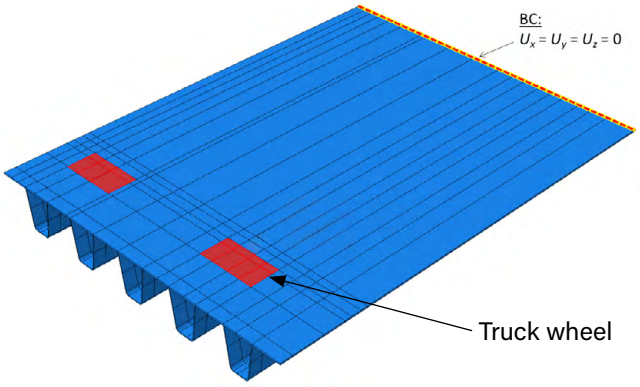


Figure 17: Isometric view of modelled deck — Boundary Condition (BC) 2: $U_x = 0$, $\theta_z = 0$.

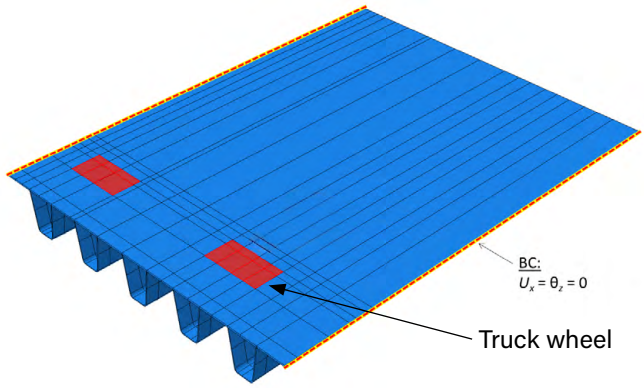


Figure 18: Isometric view of modelled deck — Boundary Condition (BC) 3: $U_z = 0$, $\theta_x = 0$.

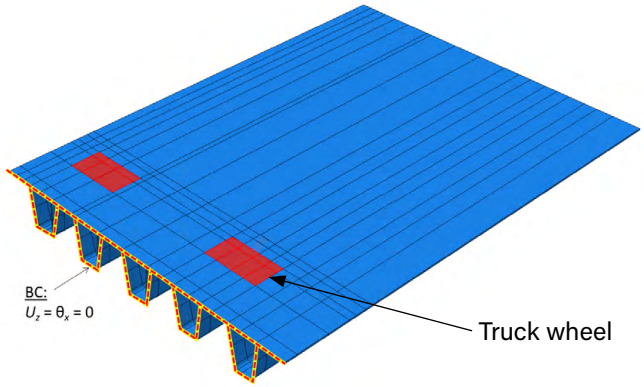
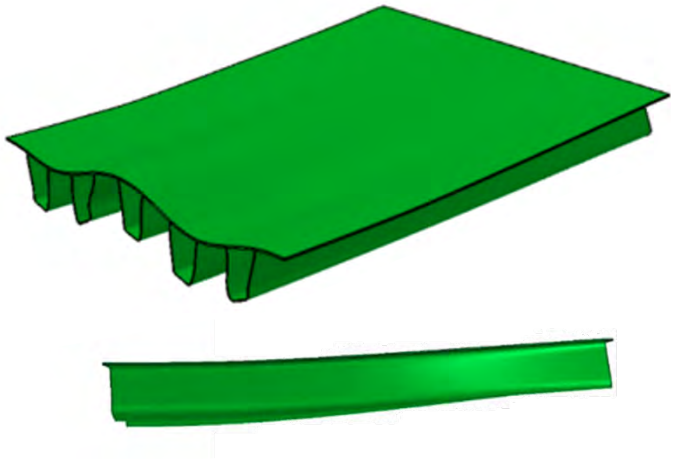


Figure 19: Isometric view (left) and side view (right) of deformed shape of deck.



4.5.4 Transverse Loading Study

Fatigue cracking of the weld can occur either at the root or at the toe depending on the inherent geometry of the deck and the transverse location of the wheel load. To demonstrate this, another fatigue study was conducted on the deck by evaluating the weld stresses at the toe and root after placing the wheel load at three locations: 1) centered on top of the rib, 2) centered between the ribs, and 3) centered on one of the deck plate-to-rib welds. These three cases are illustrated in Figure 20.

Figure 20a: Case 1 loading scenario with load centered on top of rib.

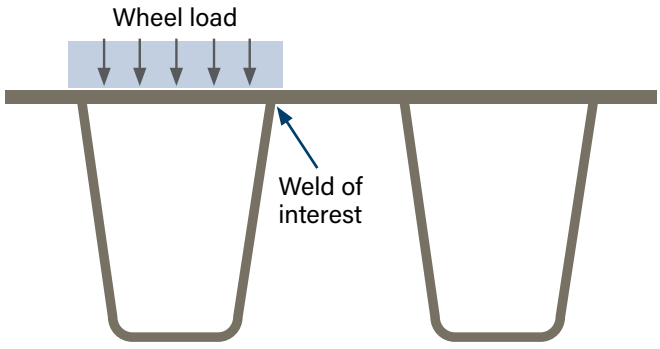


Figure 20b: Case 2 loading scenario with load centered between the ribs.

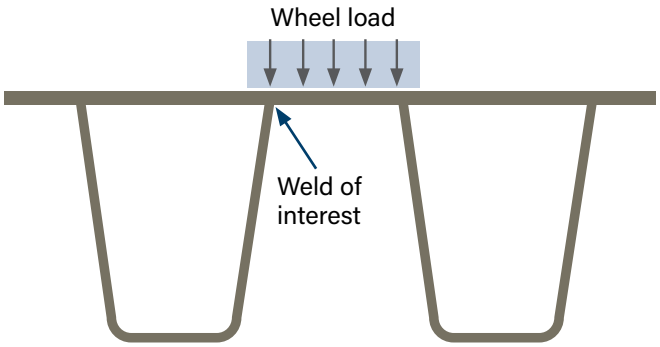
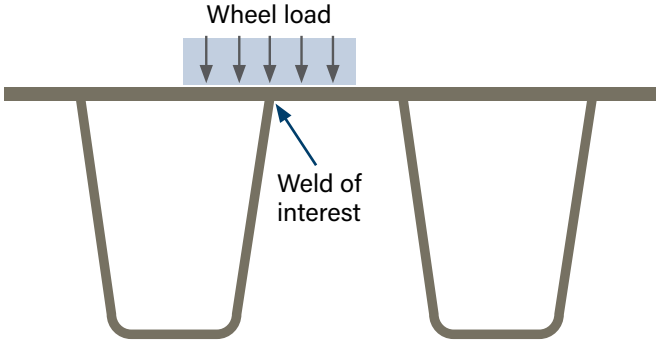


Figure 20c: Case 3 loading scenario with load centered on top of weld.



Based on the IIW recommendations [24], the hot-spot stress method should not be used to analyze the weld root, and should only be used to analyze the weld toe. The fatigue curves used to analyze the toe are FAT 100 [24]. However, two studies [13], [48] present examples of this method being used to successfully analyze the root as well as the toe. The purpose of the investigation conducted for this report was to determine which loading scenario maximizes the stress at the toe and which one maximizes the stress at the root. For the transverse loading study, all other modelling assumptions remained unchanged, with only the location of the wheel load patches varying transversely between analyses.

5 Results and Discussion

5.1 Deflection Analysis Results

5.1.1 3D FE Analysis

Table 4 displays the extreme relative deflection between ribs with the CSA S6 and AASHTO truck scenarios. The results indicate that the AASHTO-specified deflection limit (2.54 mm) is slightly exceeded with both trucks. Comparing the Canadian and US results, a deflection of 3.37 mm is obtained with the Canadian truck, which is 9% higher than the US truck at 3.10 mm.

The maximum deck deflection results are compared in Table 5. The deck deflection limit was L / 300, which is 30 mm for an L = 9.0 m span. The investigated deck panel meets this requirement in all cases. For comparison, the 3D FE (Abaqus) results are also presented with results obtained using the 2D SAP2000 model. From this comparison, the deflections obtained using the 3D FE model are 36 to 40% higher than the ones obtained using the 2D model. While caution should be made in generalizing results from analyzing only one deck system, these results highlight the importance of using a 3D model to accurately predict deflections, given the complex two-way behaviour of these deck systems as well as the complex local deflections possible at the points of support and load introduction.

Comparing the Canadian and US results obtained using the 3D FE model, the Canadian truck deflection is 22% higher. The 2D SAP2000 results are similar, showing a 25% difference in deflection, confirming that because of the elastic nature of the analysis, the 2D SAP2000 model is sufficient for studying relative differences in deflection obtained using the CSA S6 and AASHTO design trucks for a broader range of spans.

5.1.2 2D Influence Lines Analysis

To extend the results to a broader range of deck system geometries, the 2D SAP2000 model was used for a simple study where the span of the panel was varied between 3 and 27 m. Table 6 shows that the difference between the Canadian and US deflections varies between 25 and 54%. Note that for this exercise, the absolute deflection values matter less

Table 4: Extreme Relative Deflection Between Ribs due to Canadian and US Truck Loads

	Canadian Truck Load	American Truck Load	CAN/US
Rib-Rib Δ (mm)	3.37	3.10	109%

Table 5: Maximum Deck Deflection due to Canadian and US Truck Loads Based on 2D and 3D Analysis

	Canadian Truck Load	American Truck Load	CAN/US
Maximum Δ (2D, mm)	11.77	9.43	125%
Maximum Δ (3D, mm)	16.05	13.20	122%
2D/3D	136%	140%	

Table 6: Varying Span Longitudinal Deflection Analysis Results

	Maximum Δ (2D, mm)		
Span	**Canadian**	**American**	**CAN/US**
3 m	0.341	0.244	140%
9 m	11.77	9.43	125%
27 m	516.2	334.7	154%

than the differences, because the deck panel was not redesigned for each span. Interestingly, the difference is greater for the longest and shortest spans. The maximum deflection for the 3 m span is caused by the single axle load of 142 kN at midspan for the US truck and by the two 125 kN axle loads for the Canadian truck. At the longest span (27 m), all axles of both design trucks must be considered. Therefore, both the axle weight distribution and the gross vehicle weight (GVW) play a significant role on the deflection analysis. The results show that the Canadian truck causes higher deflections for all three spans, with the largest difference being 54% for the longest span.

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5.2 Fatigue Analysis Results

5.2.1 Comparison of Canadian and American Design Stress Ranges

Given that the resistance curves for the various fatigue detail categories in CSA S6 and AASHTO are identical, any differences between fatigue life predictions made using the two codes would be due to differences in the calculated design stresses or stress ranges. The results of the comparison using the sample OSD system are summarized in Table 7.

Given that the Canadian axle tandem is significantly heavier (250 kN vs. 141 kN) and the tire load configurations are similar, it is not surprising that the

Table 7: Fatigue Comparison Results

Hot-Spot Stress at Weld Toe (MPa)			
Load Case	**Unfactored σ-toe (MPa)**	**Factored σ-toe (MPa) – Finite**	**Factored σ-toe (MPa) – Infinite**
CAN	29.0	23.4	18.9
US	16.4	15.0	14.0
CAN/US	177%	155%	135%

ratio of the unfactored stresses is also high (177%). To convert the unfactored stresses into the factored stresses needed for design, factors must be applied for finite or infinite life design (i.e., design using the sloping part of the S-N curve for low volume roads or the fatigue limit for higher volume roads). These factors are summarized in Table 8, where a load factor *(load)* modifies the unfactored load and thereby the stress range proportionately, a dynamic load allowance *(dynamic)* modifies the unfactored stress range to consider dynamic (impact) effects, a reduction factor *(reduction)* is applied to the stress range to account for differences between the design truck and the real traffic (e.g., related to damage equivalence in finite life design or extreme loads for infinite life design), and OSD factors *(OSD)* specifically modify the design stress range for OSDs. The factored stress is obtained by multiplying the unfactored stress by each factor successively in the relevant column in Table 8. The resulting factored hot-spot stresses are listed in Table 7.

Table 7 shows that the Canadian load factors result in higher factored (design) stress ranges than the US load factors, with the Canadian design stress range being 55% greater for finite life design and 35% greater for infinite life design. Evaluation of the appropriateness of this difference is difficult without access to databases of current real traffic characteristics in Canada and the US, which are required to determine whether Canadian traffic is more damaging from a fatigue perspective. As shown in Figure 21, historical data and analysis suggest there are differences in the GVW spectra between Canada and the US that justify the difference in the fatigue design stress range [54].

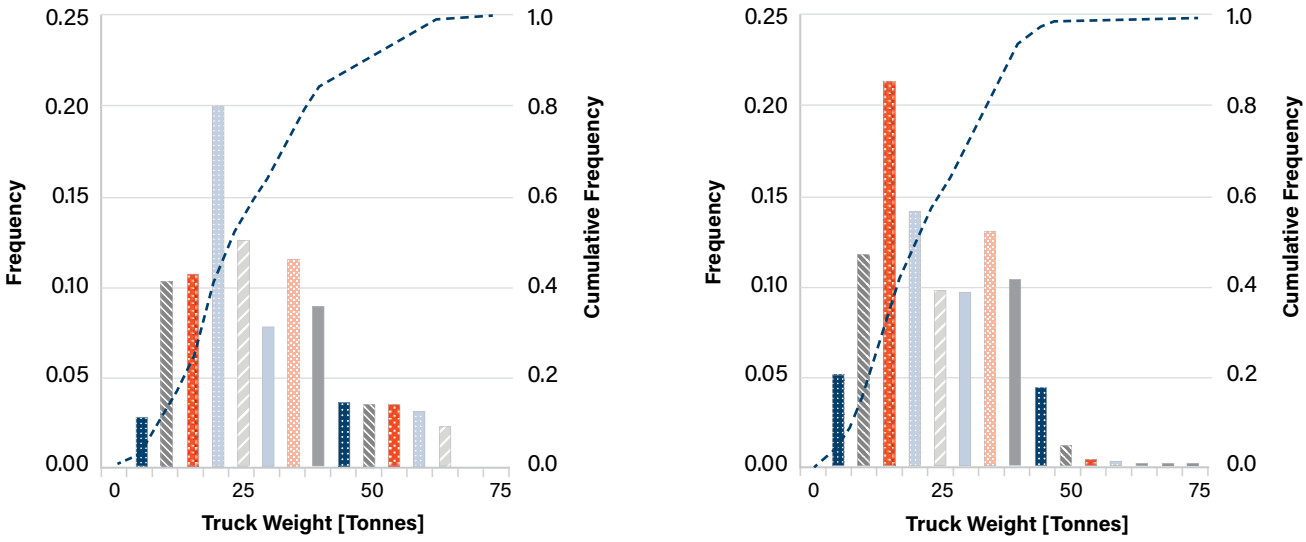
Perhaps the most valuable result of conducting this comparison was the insight gained concerning the lack of clarity in how the CSA S6 OSD fatigue provisions should be implemented. One reason for this is that AASHTO presents damage equivalence (0.8) and infinite life threshold offset (1 / 1.75 = 0.571) factors as load factors, which are prescribed in Chapter 3, where loads and load factors are described. Conversely, CSA S6 presents these factors (0.62 for finite life damage equivalence and 1/2 = 0.5 reduction applied to the long-life fatigue limit) as corrections embedded in the limit state functions themselves. In interpreting CSA S6, a designer who does not fully appreciate the intent of these factors could easily get confused and apply the US factors with the Canadian design tandem or apply the American adjustment factors in addition to the Canadian ones, leading to either unsafe or overly conservative design calculations.

Table 8: Fatigue Load Factors in CSA S6 and AASHTO

Finite Fatigue Life		
Factor Type	**CSA**	**AASHTO**
Load	1.0	0.8
Dynamic	1.3	1.15
Reduction	0.62	—

Infinite Fatigue Life		
Factor Type	**CSA**	**AASHTO**
Dynamic	1.3	1.15
OSD	—	1.3
Reduction	0.5	0.571

Figure 21: Comparison of GVW histograms from Ontario (left) and the US (right). Adapted from [54].



5.2.2 Transverse Loading Study

Table 9 presents the results of the transverse loading study, carried out to determine the importance of the wheel line location on the criticality of the weld toe and root from a fatigue perspective. The results demonstrate that Case 1 (Figure 20a, load is centered over the rib) results in a maximum toe stress 84% higher than that of Case 2 (Figure 20b, load is centered between the ribs), and 5% higher than that of Case 3 (Figure 20c, load is centered on one of the deck plate-to-rib welds). Case 3 results in the highest stress at the root, and is 42% higher than those of load Cases 1 and 2.

Only qualitative conclusions can be drawn from the comparisons between the root and toe stress as no fatigue curve has been recommended for the hot-spot stress at the root of the weld. A notch stress approach is recommended in AASHTO for such a comparison, but no notch stress design curve is given. To find such a design curve, the IIW recommendations [24] could be referenced.

Table 9: Varying Transverse Locations of a Canadian Design Truck Load Showing Weld Stresses at the Toe and the Root

Hot-Spot Stress (MPa)			
	Case 1	Case 2	Case 3
σ-Toe	30.5	16.3	29.0
σ-Root	21.7	21.8	31.0

6 Gross Vehicle and Tandem Axle Weight Data Analysis

For fatigue verification of an OSD system, the calculated fatigue stress range is taken as that due to the passage of a tandem set of 125 kN axles, spaced 1.2 m apart (with a transverse spacing of 1.8 m). A comparison of tandem axle weights across provinces would be of interest in this context, given that some provinces design bridges with a heavier design truck, but these provinces do not currently use a correspondingly heavier tandem axle set for the local check of bridge deck details. For example, Alberta uses the CL-800 truck instead of the CL-625 truck used in other provinces, and Ontario uses a modified CL-625-ONT truck. Two databases of heavy vehicles, provided by the highway ministries from each province, have made it possible to compare GVWs and tandem axle weights, as shown in Figure 22. It should be noted that the database from Alberta contains weigh-in-motion data collected from 2004 to 2022 at five locations across the province (more than 62 million vehicles over 10 tonnes), whereas the database from Ontario is from a 2012 survey of vehicles that were stopped and weighed and contains approximately 40,000 vehicles over 10 tonnes.

Figure 22: Comparison of GVW and tandem weight histograms in Ontario and Alberta.

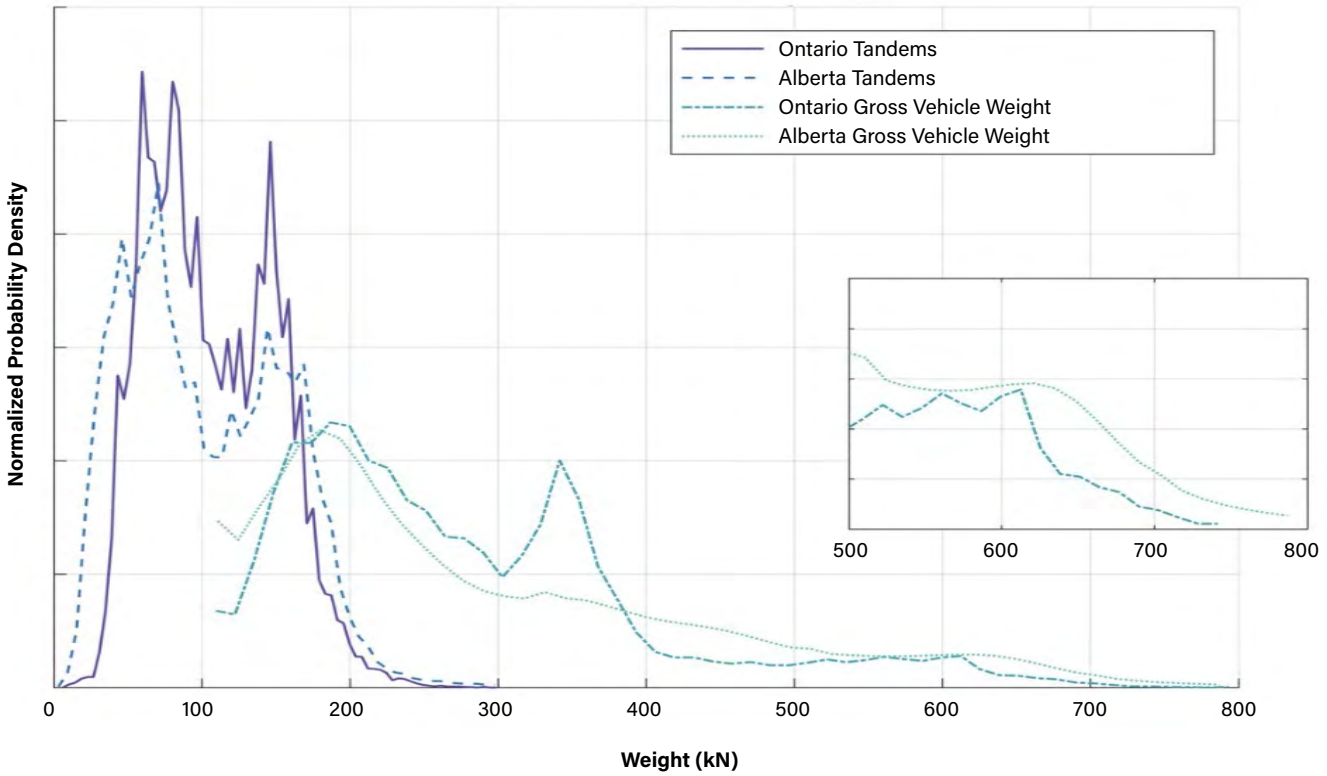


Figure 22 provides a comparison of total GVW and total tandem axle weight in Ontario and Alberta. Alberta has heavier vehicles compared to Ontario, as seen in the longer, thicker tail past 600 kN. The local maximums corresponding to empty and full vehicles are more obvious in Ontario, while full vehicles in Alberta are distributed over a broader range of weights.

According to Figure 22, the tandem axle weights are not markedly different between the two provinces. However, Table 10 shows that Alberta has higher GVWs at high percentiles, despite the GVWs being lighter than Ontario at the median (50th percentile). Care should be taken when interpreting extremes because the Alberta data is from weigh-in-motion, which is more prone to scatter when compared with the static weights from the Ontario survey. It should also be noted that for these calculations, axles above 300 kN (more than double the legal limit) and below 30 kN are excluded to ensure an apples-to-apples comparison to the greatest extent possible, and also because the axle loads above 300 kN may be erroneous measurements in some cases.

From a fatigue perspective, the extreme percentiles may not be as important as they would be in an ULS verification. A simple calculation using both datasets and a fatigue curve with slope $m = 3$ and no endurance limit indicates that when the axles are scaled to give a damage index of 1.0 for the Ontario tandems (using Miner's sum), the corresponding damage from Alberta is 15% higher at 1.15 (if damage is adjusted for the same number of vehicles in each province; see Table 11). The top row of Table 11 shows fatigue damage resulting from two code truck tandems and two provincial tandem datasets, normalized by the damage index from the provincial data in Ontario.

The same calculation using gross vehicle weights in both databases reveals 36% more damage from the Alberta vehicles. This simple calculation is irrespective of bridge geometry and assumes one stress cycle per vehicle equal to the scaled vehicle weight. The result indicates that while Alberta vehicles are heavier overall, and more damaging from a fatigue perspective, this is not reflected in the tandem axle groups to the same degree. This suggests that the increased weight of Alberta vehicles is probably spread over more axles.

Table 10: Comparison of Gross Vehicle Weights and Tandem Weights in Ontario and Alberta

Percentile	Ontario Tandems (kN)	Alberta Tandems (kN)	Ontario GVW** (kN)	Alberta GVW (kN)
50	100.1	99.1 (1% ↓)	248	238 (4% ↓)
95	176.6	188.4 (7% ↑)	558	618 (11% ↑)
99	206.2	224.6 (9% ↑)	649	722 (11% ↑)
99.9	246.6	282.5 (15% ↑)	745	898 (21% ↑)

Note. *Alberta data is from weigh-in-motion and therefore subject to greater scatter. ↓↑ arrows indicate direction of difference between Ontario and Alberta result. **GVW = gross vehicle weight.*

Table 11: Relative Damage due to Ontario and Alberta Tandem Axles (top) and Trucks (bottom)

	CSA S6 0.52 · 250 kN	CSA S6 0.62 · 250 kN	Ontario Tandem Damage	Alberta Tandem Damage
Tandem Axles	1.21	2.04	1.00	1.15

	CSA S6 0.52 · 625 kN	CSA S6 0.52 · 800 kN	Ontario GVW** Damage	Alberta GVW Damage
Trucks	0.94	1.98	1.00	1.36

Note. *All damage indices benchmarked for Ontario damage and adjusted for the number of tandems/vehicles. **GVW = gross vehicle weight.*

A final comparison is made between the tandem axle damage indices from each location and that of CSA S6. The latter was taken as 0.62 multiplied by two 125 kN tandem axles for a total of 155 kN. Damage from the passage of the code tandem, scaled for the number of passages to compare to the other databases, gives 2.05 or a 105% increase. All calculations assume both axles are summed together. The effect of the 1.2 m spacing in the CSA code tandem, compared to the larger spacing observed in both databases, as shown in Figure 23, would add further conservatism.

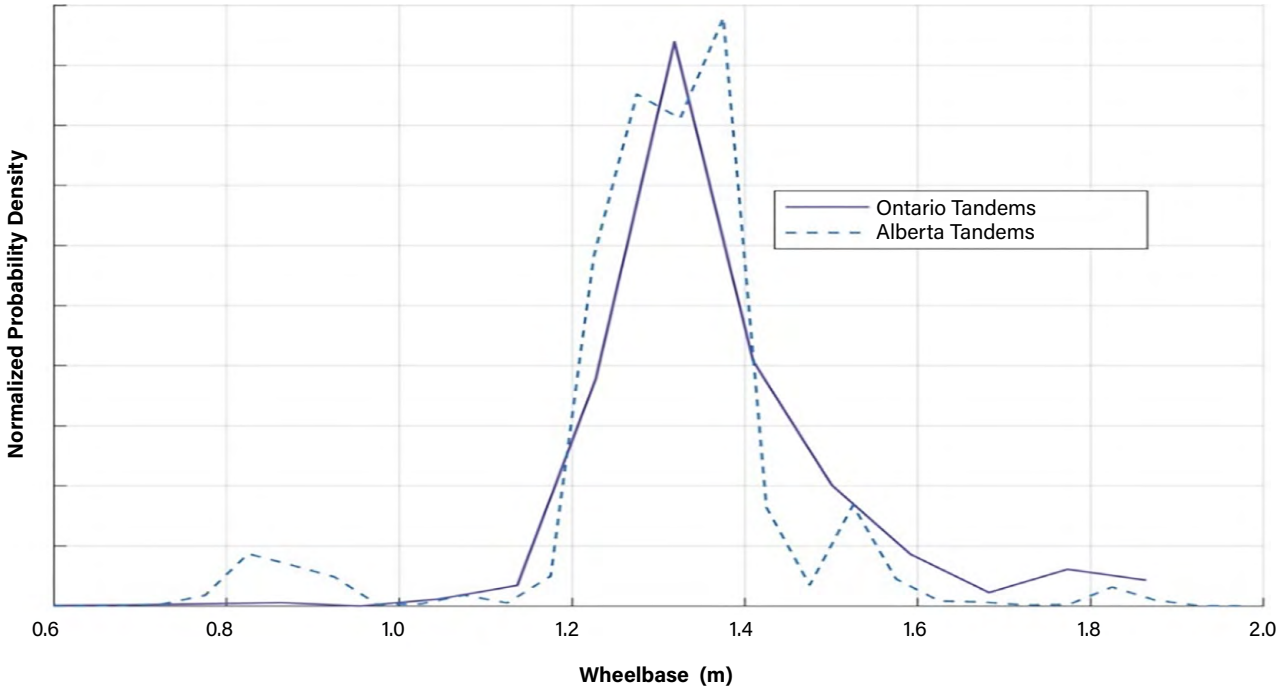
From Table 11, it could be concluded that the current CSA S6 fatigue verification is conservative in requiring a damage equivalence factor of 0.62 for decks and scaling the fatigue design stress proportionally to the truck weight for ULS design. However, if one were to assume the current fatigue design check is appropriate in Ontario and if a 35.7% increase in fatigue damage were sought from the CSA S6 code truck, then only an increase from 625 to 692 ($= 1.357^{1/3} \cdot 625$) would be needed. This means something lighter than 800 kN (such as a CL-700) could be used for fatigue design in Alberta. Similarly, if a 14.7% increase in fatigue damage were sought from the CSA S6 125 kN tandem set, then only an increase from 125 to 131 ($= 1.147^{1/3} \cdot 125$) would be needed. While this suggests a heavier tandem is needed in Alberta, it could be argued that the 125 kN tandem is not too unconservative (it is less than 5% below where it should be) and is therefore acceptable to retain in the interest of simplicity.

7 Conclusions

Based on the research presented and discussed in this report, the following conclusions may be drawn:

- While a considerable body of knowledge exists on the fatigue behaviour of OSDs, several gaps remain, including details related to the verification of the rib-deck weld root. Specifically, using the notch stress approach and the application of a performance-based approach for assessing the consequence of not meeting penetration targets when fabricating the rib-deck weld.

Figure 23: Comparison of Ontario and Alberta tandem axle wheelbases.



- Designer experience is invaluable for a successful OSD design. An appreciation of durability and constructability issues is critical, recognizing that optimal designs are not usually those that simply minimize material requirements. For example, more durable and economical designs often result from using a higher deck plate thickness to reduce deflections and fabrication cost.
- The approach prescribed in CSA S6:19 results in higher calculated deflections for OSDs than if a designer uses AASHTO on its own. However, this difference may be justified, given the higher truck and axle loads that can be present in Canadian traffic.
- The approach prescribed in CSA S6:19 results in higher calculated design fatigue stress ranges for OSDs than if a designer uses AASHTO on its own. However, this difference may also be justified, given the more severe nature of the Canadian truck population.
- Implementation of the approach prescribed in CSA S6:19 for fatigue design of OSDs could result in unsafe or overly conservative errors by designers who are not familiar with the purposes and sources of the various required factors (i.e., load factors, damage equivalence factors, and dynamic load allowances), due to a lack of specificity regarding when the different factors apply.
- Regional extreme GVW differences, which might lead to the specification of different CL-W trucks for ULS design, may not accurately reflect regional differences in the damage characteristics of the overall truck population. In addition, they may also exceed the regional differences in the critical tandem axle loads for ULS and fatigue design. Thus, while it may be desirable to consider regional (i.e., interprovincial) differences in load characteristics in design, when it comes to the fatigue design of elements with short influence lines like OSDs, it may be overly conservative to do this based solely on a comparison of the extreme GVWs relevant for ULS design.

8 Recommendations for CSA S6:25

The literature review, industry survey results, results of the deflection and fatigue analyses, and other input received during this project (e.g., input from the Project Advisory Panel and CSA S6 Committee members) informed the following recommendations for improvements to OSD provisions that should be incorporated into CSA S6:25.

Recommendation 1: Conflicting language between CSA S6 and AASHTO needs clarification.

Conflicting language in CSA S6 and the AASHTO design specification should be clarified. For example, there are slightly different minimum plate thickness requirements prescribed in each code. Because these requirements pertain mainly to durability against corrosion, the CSA S6 minimum thickness (6 mm) is likely more appropriate for Canadian applications. Based on feedback from fabricators, selecting the smallest required thickness to meet strength requirements does not always lead to the most economical design, so this requirement should not be prohibitive from the perspective of ensuring that cost-effective OSDs can be designed in Canada.

Recommendation 2: Deflection limits should be maintained despite the heavier design truck in CSA S6.

While applying the AASHTO deflection limits with the CSA S6 design truck may lead to a requirement for stiffer deck systems (10–50% stiffer, depending on the span and the relevant deflection requirement), these deflection limits should be maintained. The basis for this recommendation is that the publicly available truck GVW data from the US and Canada, as well as more recent unpublished data not included in this report, suggests there is a significant difference between the weights of the heaviest Canadian and US trucks. Because the AASHTO deflection requirements help to ensure the durability of the deck wearing surface, there may be little basis for systematically relaxing them. However, there is a lack of recent research on wearing surface durability, and new wearing surface systems are emerging. If it can be shown that these systems can handle larger cyclic deflections, then a set of deflection limits based on wearing surface performance could be developed in the future.

Recommendation 3: CSA S6 OSD fatigue design should use the Canadian tire footprint.

Allowing OSD fatigue design in CSA S6 to use the Canadian tire footprint used for ULS design (600 mm x 250 mm) rather than the US one could provide a significant benefit because this would allow the entire design to be conducted with one FE model, avoiding the need for remeshing between the ULS and SLS design checks and the FLS design check.

Recommendation 4: Fatigue design should be based on CSA S6, except for some modelling aspects.

To increase clarity regarding fatigue design stress range calculations, fatigue design should be done according to CSA S6, except for information on modelling aspects (e.g., effective flange width calculation) and resistance categories of various fatigue details, for which AASHTO should continue to be referenced. The AASHTO fatigue detail categories (A, B, C, etc.) use similar S-N curves to the CSA S6 categories, whereas the dynamic load allowance, damage equivalence factor, and reduction factor for infinite life design have been established based on Canadian research and calibration efforts and are therefore more appropriate for Canadian traffic.

Recommendation 5: CSA S6:25 should add special fatigue detail category (C*).

AASHTO has one special fatigue detail category (C*) for OSDs for which the resistance S-N curve is defined using a special function. CSA S6 should incorporate this function so this fatigue detail can be easily addressed using the recommended new design approach.

Recommendation 6: Detail Category A should be used for design using the notch stress approach.

For fatigue verification of the weld root in the PJP deck plate-to-rib weld, AASHTO suggests using the notch stress approach, but no resistance curve is defined. A comparison of the design curve recommended by the IIW [24] and the CSA S6 Detail Category A shows that the latter is slightly conservative. It is therefore recommended that Detail Category A be used for the notch stress approach.

Recommendation 7: An entry is needed for CSA S6 Table 10.5 for OSD fatigue design.

To use CSA S6 for fatigue design of OSDs, a new entry is needed in Table 10.5, which prescribes the design number of cycles, N_d, per truck passage. For this value, it is recommended that CSA S6 use the corresponding value in AASHTO of 5.0.

9 Future Research Needs

Based on the research presented and discussed in this report, the following future research needs were also identified:

The impact of deflection on durability of the wearing surface requires further investigation.

Research on the effect of deflection on the durability of the wearing surface is warranted, with a focus on newer wearing surface systems and the Canadian climate, as well as other geographic issues, such as the effect of snow removal, de-icing salt, and so on.

More traffic data is needed not only for OSD research but also for future code development.

A more detailed investigation of the available weigh-in-motion data from various regions of Canada would be beneficial for better understanding the safety levels provided by the tandem axle group currently used for OSD fatigue design. On a broader scale, the same questions could be asked about the current practice of scaling the fatigue design truck with the ULS truck. This research would benefit from the support of transportation ministries across Canada.

New OSD systems for medium-span bridge applications should be investigated.

Exploring the development of a new OSD system optimized for medium-span bridge applications would be beneficial as a potential solution for rapid bridge construction, emergency bridges, and prefabrication of bridges to be installed in remote locations. This would likely require that the OSD system work compositely with the steel girders to achieve an economic design.

A performance-based approach for LOP assessment would be beneficial.

Further research to improve our understanding of how level of weld penetration impacts the fatigue performance of the deck plate-to-rib weld would be beneficial. Given the difficulty of achieving a precise LOP and assessing LOP using non-destructive evaluation methods, a performance-based approach that informs the designer of when an achieved LOP is acceptable from a fatigue perspective could help to improve durability and reduce fabrication cost.

This research should be extended to other fatigue details in OSD systems.

A state-of-knowledge review and investigation of the fatigue performance of weld details common to OSDs other than the rib-to-deck plate weld, such as the complex connection details that are necessary around the floor beam-to-rib connections, may be warranted.

Further development of Canadian OSD design provisions and expertise is needed.

Ongoing efforts are recommended to build expertise in Canada on OSD design with the goal of developing a full set of design provisions for the next CSA S6 revision cycle. These efforts would help to ensure that region-specific issues related to fabrication, environmental service conditions, traffic loading, and so on, are adequately addressed in future code editions.

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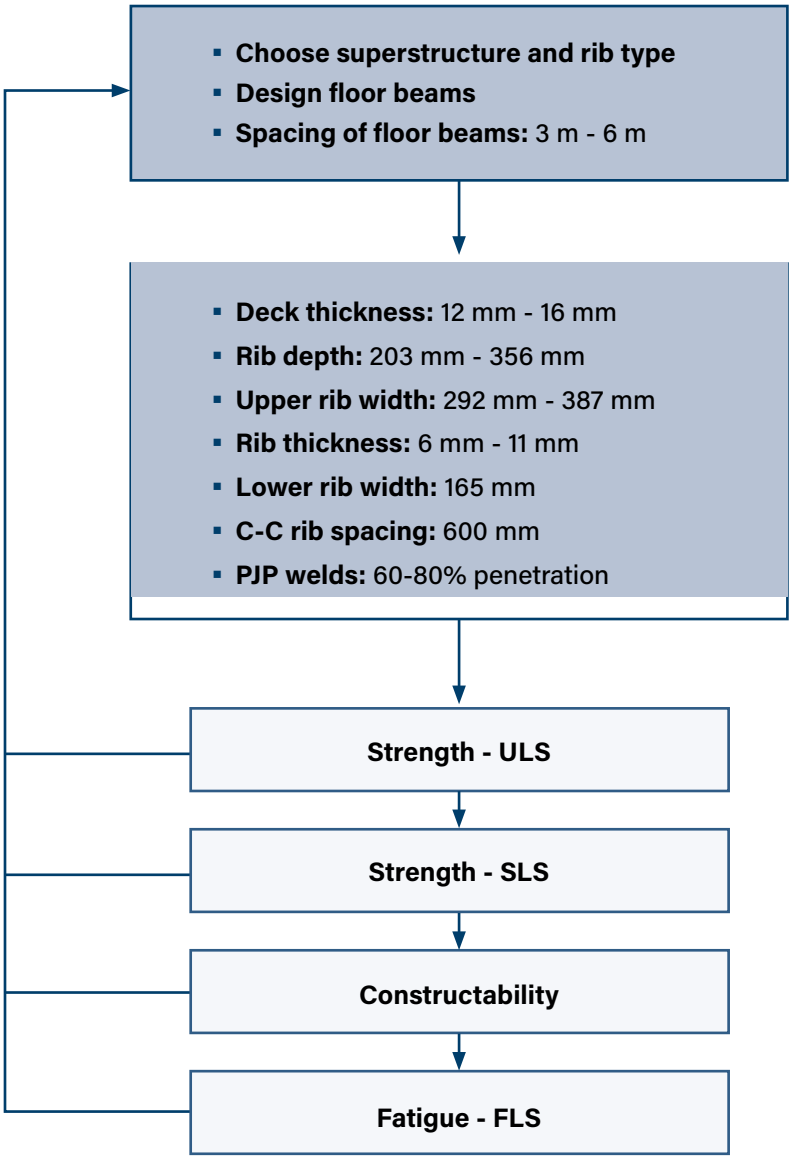
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Appendix 1 - Proposed OSD Design Flow Chart



Note. *OSD design flow chart adapted from [1].*
Single-cell closed cross-sections provide best solution to facilitate inspection of rib-deck welds.
Increasing floor beam spacing requires fewer costly rib-to-floor beam intersections. It is difficult to control the fatigue stresses at these intersections for long rib spans.
Larger deck thickness improves fatigue performance, 16 mm being optimal thickness.
Using larger overall ribs improves fatigue performance.
Larger rib thicknesses reduce risk of blow-through/melt-through and promote consistent welding.

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