Discussion on the Development and Jurisdictional Implementation of Overhead Sign Support Structure Designs and Codes

T.A. Epp, Structural E.I.T., Dillon Consulting Limited

D.R.C. Amorim, M.Sc., P.Eng., Associate, Dillon Consulting Limited

S.S. Rihal, M.Eng., P.Eng., Associate, Dillon Consulting Limited

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Abstract

Overhead sign support structures (OHSS) are a significant infrastructure item with 231 in service in Winnipeg and an additional 105 under the jurisdiction of the Province of Manitoba as of 2016. Proper design of these structures is paramount because of their proximity to roadways and the risk to commuter safety should they fail. This paper endeavours to provide a review of the advancements made in the governing codes (specifically the *AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals* (AASHTO Specifications); review practices across various jurisdictions with respect to selecting and implementing fatigue categories from the various editions of the AASHTO Specifications; and finally review a Manitoba case study covering a significant update to the Standard Designs for Traffic Signals.

The current 2015 AASHTO Specifications suggests designing OHSS for a minimum 50 year design life and the Specifications have evolved through time to provide the tools necessary to design structures that provide this minimum service life. One of the developments that enables the design of OHSSs with the expectation of 50 years of service is the improved consideration of the fatigue limit state. The combination of the large surface area of the attachments and the long, thin structural members of the OHSS increases fatigue stresses initiated by wind loading in the form of natural wind gusts, truck-induced gusts, and galloping fatigue, making these important considerations when designing the structures. As recently as the 1994 edition, the AASHTO Specifications referenced the *AASHTO Specifications for Highway Bridges* for fatigue design of OHSSs. The Specifications have evolved markedly since 2001 when fatigue requirements were first implemented as a stand-alone chapter, independent of the fatigue requirements in the Specifications for Highway Bridges.

The AASHTO Specifications provide recommendations for the fatigue category (essentially an importance factor) selection based on considerations such as the amount of traffic and traffic speed on the roadway where the OHSS is to be installed. Ultimately, however, the decision rests with the jurisdiction that governs the location to select the fatigue category. This has led to varying design practices across jurisdictions depending on several factors, including how the structures have performed historically in that area, the perception of how conservative the designs that stem from each of the three fatigue categories are, and the consideration of the initial cost of the structure.

1.0 Introduction

Overhead sign support structures (OHSS), luminaires, and traffic signal structures are ubiquitous in the built environment. To the casual observer, these structures serve to simply convey directions and information during commutes and travel, light the roadway, and direct traffic. In actuality, these structures require significant design effort and have a dedicated section in the CAN/CSA-S6 Canadian Highway Bridge Design Code (CHBDC) and a standalone specification in the form of AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals. The design considerations include service, fatigue, strength, and extreme limit states. The considerations have changed over time to reflect knowledge gained through experiment and spurred occasionally by failure. The ubiquitous nature of these structures can make one lose sight of the fact that they are hazards in the sense that they can fail and fall into the travel lane; additionally, the sheer size of the structures necessary to span significant distances and carry large signs results in roadside hazards that must be adequately protected with crash protection or otherwise positioned beyond the clear zone. Public safety is the primary reason to further investigate the way in which design has been, and is being carried out, and to improve the practice where possible.

For brevity, the three structures identified in the AASHTO Specifications (OHSS, luminaires, and traffic signals) will be jointly referred to as roadside structures and parsed out when independent discussion of each structure is warranted. There are many different approaches to roadside structure design as discussed in Section 3.0 Review of Jurisdictional Practices. Different Departments of Transportation (DOTs) have implemented design practices to different editions of the updated specifications. There is further variance in how certain aspects of the specifications are interpreted. In particular, fatigue categories, mean return intervals for wind speed, and the use of mitigation devices can lead to different physical structures even though the specifications used as the basis for design may be the same. One of the aspirations of this paper is to take a broad look at the various design practices across DOTs to learn from how others practice and study the range of design practices that have been developed.

One of the drivers in updating the code and design methodology is the difference in performance and conformance to changes. Manitoba's DOT – Manitoba Infrastructure (MI) – recently completed the process of updating their traffic signal structures and advanced warning sign designs to the latest AASHTO Specifications; this was done to ensure the designs are up-to-date and to ensure that recent concerns regarding structure service life and performance are not perpetuated through new structure installations being designed to superseded standards. A case study is presented comparing the physical dimensions of similar structures designed to different specifications and examining the influence that factors such as fatigue importance category designation and wind speed play in overall member sizing.

An area of interest for roadside structures is the use of vibration mitigation devices on the structures. To date, mitigation devices have not found widespread use across DOTs. The implementation of these devices does however warrant investigation as the design criteria can be reduced when they are successfully implemented and therefore significant savings in material costs can be realized. The discussion on mitigation devices in Section 3.0 Review of Jurisdictional Practices, is expanded on with a case study examining the reduction in section properties of a structure designed for use with and without a mitigation device.

2.0 Development of Standards for Roadside Structure Design

The primary standards used to design roadside structures in the United States and Canada are the AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals and the CHBDC, respectively. The AASHTO Specification was updated from working stress design (WSD) to a load and resistance factor design (LRFD) approach in 2015 whereas the the CHBDC has used LRFD since prior to 2006. In general, WSD involves determining the stress in an element and comparing it to the allowable stress – a methodology otherwise known as allowable strength design (ASD). In load resistance factor design (LRFD), the load and resistance are factored and compared to determine the adequacy of the section. One of the differences between the two methods is way in which the efficacy of the design is tested for strength and serviceability. For WSD, arbitrarily defined factors of safety are used to determine if the applied stress is sufficiently below the allowable stress, whereas LRFD uses resistance factors for both loads (typically factored up) and materials (typically factored down) before comparing the applied load to the member resistance.

The approach to roadside structure design in the CHBDC has not significantly changed since the 2000 version with the corpus of the material included in the code remaining unchanged. The AASHTO Specifications, on the other hand, have undergone significant changes not only to the design methodology but also to the approach to fatigue considerations in recent updates.

2.1 Development of Fatigue Design Requirements in the AASHTO Specifications

Although the member design sections of the AASHTO Specifications were updated to LRFD in 2015, the fatigue considerations remained based on determining the stress applied to each component. Fatigue is typically considered as a response to stresses and stress concentrations and therefore, it is likely that fatigue design will not deviate in the foreseeable future. The consideration of fatigue in design in the AASHTO Specifications has however evolved significantly in the recent past. An overview of the changes and current approach of the AASHTO Specifications is provided in the subsequent sections.

2.1.1 Fatigue Design Requirements Prior to AASHTO Specification (2015)

Fatigue is the progressive and localized structural damage that occurs when a material is subjected to cyclic loading and the stress fluctuations that accompany the loading. The design of roadside structures has experienced several code changes with respect to fatigue over the last two decades, particularly in relation to the understanding of fatigue behaviour and mitigation of fatigue behaviour of the structures.

The 1994 AASHTO Specifications referenced the AASHTO Specifications for Highway Bridges as the basis for fatigue design of roadside structures. The 1994 AASHTO Specifications suggested that the infinite fatigue life of the material be used in fatigue design; for steel members, this was taken as two-million cycle fail stress. As per the AASHTO Specifications for Highway Bridges at the time, the ultimate strength of steel roadside structures was compared to the load effects caused by the maximum expected wind event – typically a one-in-fifty (1-in-50) year wind event was used for the design.

The 2001 AASHTO Specifications was the first edition to outline fatigue requirements specifically for overhead sign support structures, luminaires, and traffic signals. The fatigue design of cantilevered structures in the 2001 AASHTO Specifications was based initially on National Cooperative Highway Research Program (NCHRP) Report 412. At this point in time, noncantilevered (bridge) structures had not been fully investigated and research was still underway to better understand the effects of fatigue on those structures [1]. The 2001 AASHTO Specifications also introduced Fatigue Importance Factors (I_F) to be determined by the Owner to adjust the structural reliability based on various conditions including but not limited to: wind speed, traffic volume, route importance, site location, and arm length.

Due to the inherent difficulties in designing for a finite design life for fatigue, the 2001 AASHTO Specifications recommended an infinite life fatigue design approach, similar to the 1994 version. In the 2001 AASHTO Specifications, however, the infinite-life fatigue approach was based on the concept of the constant amplitude fatigue limit (CAFL). The CAFL is defined in the AASHTO Specifications as the nominal stress range below which a particular fatigue detail can withstand an infinite number of repetitions without fatigue failure. According to the AASHTO Specifications, the structure should be designed so that the stress induced by the various fatigue types (truck-induced, natural wind, galloping) is below the calculated stress at the CAFL. The 2009 (5^{th} Edition), 2013 (6^{th} Edition) and 2015 LRFD-Interim Specifications all follow the CAFL design approach in addition to improvements in fatigue-prone connection details through ongoing research and experimental studies such as the NCHRP Report 494 that was used to inform fatigue design considerations in the 2001 specifications.

2.1.2 Current Fatigue Design per AASHTO Specifications (2015)

The 2015 AASHTO Specifications are the latest specifications for the design of roadside structures. The three possible fatigue cases considered per the 2015 AASHTO Specifications are fatigue due to natural wind gusts, truck induced wind gusts, and galloping. Vortex shedding is also discussed but no importance factors are provided in the current 2015 AASHTO Specifications. In general, vortex shedding is typically only considered for high-mast lighting towers and is incorporated into the equivalent static combined wind pressure range [2]. Further to the three fatigue cases (natural wind gusts, truck induced gusts, and galloping), there are three Fatigue Categories that modulate the calculated stress in the element being analyzed through the application of the corresponding Fatigue Importance Factor, I_F. Category I represents the most severe, Category II is moderate, and Category III represents the least severe case. The calculated factored stress for each of the three fatigue cases is compared to the CAFL specified in the AASHTO Specifications for the component or detail and the fatigue life of the element is said to be infinite when the calculated factored fatigue stress is less than the given CAFL.

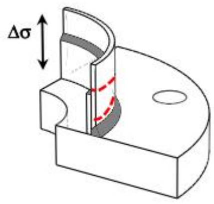
4.7 Full-penetration groove-welded tube-to-transverse plate connections with the backing ring not attached to the plate, and the backing ring welded to the tube with a continuous fillet-weld at top face of backing ring.	*KF≤ 1.6: 11.0* *1.6< KF ≤ 2.3: 3.9*	*KI≤ 3.0: 10.0* *3.2< KI ≤ 4.0: 7.0* *4.0 < KI ≤ 6.5: 4.5*	In tube wall along groove-weld toe or backing ring top weld toe.	Column-to-base-plate connections. Mast-arm-to-flange-plate connections. 

Figure 1: Representative Fatigue Detail from the 2015 AASHTO Specifications [2]

Figure 1 shows a detail from the 2015 AASHTO Specifications Table 11.9.3.1-1 showing one of the standard details for the weld detail of a backup strip to a shaft or arm. The table gives a written description of the detail as well as a pictorial one. The table also gives values for finite fatigue life calculation and the CAFL limit.

3.0 Review of Jurisdictional Practices

DOTs use different design specifications, experience different climatic conditions, and have developed different design practices which has resulted in the design of roadside structures varying significantly from DOT to DOT. As part of the research carried out for this paper, several DOTs were canvased for response to a survey that was constructed to determine various design practices and the basis for those practices. Approximately half of those contacted responded for a total of eleven responses available to provide insight into practice across various DOTs. This section compares and discusses jurisdictional practices in several areas such as the interpretation of the standards and decisions on areas such as fatigue category and wind speed selection based on the responses from the participating DOTs. The selection in these areas can be guided by practice, experience, and evaluation of historical performance of similar structures to those being designed as well as the impact to the cost of fabricating the structures. The following sections will identify several areas where practice could vary between jurisdictions and look at the rationale for any differences. The first area identified for each DOT is the standard used for design. The design standard used for each DOT is provided in **Table 1.**

Table 1: Design Standard used by Each Responding DOT

Location of DOT	Specified Design Standard
Province of Alberta	CHBDC
Province of British Columbia	CHBDC
Province of Manitoba	AASHTO Standard Specifications
Province of Nova Scotia	CHBDC
Colorado	AASHTO Standard Specifications
Illinois	AASHTO Standard Specifications
Montana	AASHTO Standard Specifications
New Hampshire	AASHTO Standard Specifications
Oregon	AASHTO Standard Specifications
South Dakota	AASHTO Standard Specifications

Location of DOT	Specified Design Standard
Utah	AASHTO Standard Specifications

Generally, the responding DOTs specify standards specific to the country the DOT is part of. The only outlier is Manitoba, Canada in which the AASHTO Specifications are used. This is in line with the DOT's approach to bridge design where the AASHTO LRFD Bridge Design Specifications are used in lieu of the CHBDC typical to other Canadian DOTs. The DOTs were also asked to respond whether the AASHTO Specifications (2015) had been adopted. The responses are documented in **Table 2**.

Table 2: Adoption of AASHTO Specifications (2015)

Location of DOT	Does the DOT use AASHTO Specifications (2015) for design?
Province of Alberta	No. The understanding is that the wind formulas in the LRFD refer to wind climatic data not available in the S6 Bridge Code
Province of British Columbia	No
Province of Manitoba	Yes
Province of Nova Scotia	No
Colorado	For designs that do not meet M&S Standards
Illinois	No
Montana	Yes
New Hampshire	Yes
Oregon	For monotube cantilever design
South Dakota	No
Utah	No

For the DOTs that use the AASHTO Specifications, five of eight respondents stated that the most recent 2015 AASHTO Specifications are used in some form. Two of the responses noted qualifications for the use of the 2015 AASHTO Specifications. Specifically, the Colorado DOT responded that designs that do not meet the M&S Standards (a compilation of DOT standards and standard details) are designed using the most recent specifications and the Oregon DOT stated that monutubular cantilever designs are carried out with the most recent specifications. Based on the response from the Oregon DOT, it is inferred that the bridge-type structures and/or non-monotubular cantilevers are designed using one of the older specifications.

Despite the various version of standards used by the DOTs, the design considerations remain similar from DOT to DOT in that the wind speed and fatigue considerations must be defined for design. The following sections address the basis for the design considerations for the responding DOTs.

3.1 Mean Return Interval Used for Wind Pressure Calculations

The standard used can have a significant impact on the wind speed used for the design of roadside structures. The wind speed is used to calculate the wind pressure applied to the structure. The particular standards used by respondents for wind speed selection are listed in **Table 3**.

Table 3: Specification used for Mean Return Interval Wind Speed

Location of DOT	Specification used for Mean Return Interval Wind Speed
Province of Alberta	CHBDC: Table A3.1.1
Province of British Columbia	CHBDC: Table A3.1.1
Province of Manitoba	AASHTO Standard Specs: Chapter 3 Basic Wind Speed Figures
Province of Nova Scotia	CHBDC: Table A3.1.1
Colorado	AASHTO Standard Specs: Special Wind Region
Illinois	AASHTO Standard Specs: Chapter 3 Basic Wind Speed Figures
Montana	AASHTO Standard Specs: Chapter 3 Basic Wind Speed Figures
New Hampshire	AASHTO Standard Specs: Chapter 3 Basic Wind Speed Figures
Oregon	AASHTO Standard Specs: Chapter 3 Basic Wind Speed Figures
South Dakota	AASHTO Standard Specs: Chapter 3 Basic Wind Speed Figures
Utah	AASHTO Standard Specs: Chapter 3 Basic Wind Speed Figures

The standard used for each DOT aligns with the general standard used for OHSS design. For the DOTs that use AASHTO Specifications, all of the respondents use the Basic Wind Speed Figures. The Colorado DOT specified that they use a special wind region to determine wind speed. The 2015 AASHTO Specifications state that “mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions” leading Colorado to consider the wind speed independently from the basic wind speeds provided due to the topography in the region [2].

The mean return interval (MRI) for the wind speed selection was also provided by the responding DOTs. The 2015 AASHTO Specifications provide three different MRIs for wind speed selection including 300, 700, and 1700 years. The wind speeds become more severe as the MRI increases correlating to the lower chance of the wind speed being exceeded in a fifty year period. For example, the 1700 year MRI corresponds to an annual exceedance probability of 3 percent in 50 years. **Table 4** outlines the interval used by each DOT.

Table 4: Mean Return Interval Period

Location of DOT	What mean return interval period is used for your selection of fatigue category?
Province of Alberta	1 in 50 (CHBDC)
Province of British Columbia	No response
Province of Manitoba	1700 year (AASHTO Standard Specs)
Province of Nova Scotia	As per the CHBDC (inferred to refer to Table 3.8-1)

Location of DOT	What mean return interval period is used for your selection of fatigue category?
Colorado	Yearly mean wind velocity
Illinois	No response
Montana	MDT requires 700 year (100 MPH) for all installations
New Hampshire	1700 year according to Table 3.8-1 (AASHTO Standard Specs)
Oregon	1700 year according to Table 3.8-1 (AASHTO Standard Specs)
South Dakota	50 year
Utah	10 year

From the responses in **Table 4** it can be seen that a wide range of approaches have been adopted for the return interval for wind speeds. Generally, the AASHTO Specifications provide maps of the United States with contours denoting different MRI wind speeds while the CHBDC provides a table with wind pressure data for many of the major centers in Canada. Wind pressures for both the 2015 AASHTO Specifications and 2016 CHBDC are calculated below for Winnipeg, Manitoba as a comparison for the equations and outputs. It should be noted that the maps in AASHTO do not extend to Canada and therefore MRI wind speeds for a similar region to Winnipeg in the United States has been used for the comparison. In particular, the central prairie region has been chosen as it is similar in topography.

The Canadian Code does not have fatigue considerations apart from vortex shedding. Wind loading in the CHBDC is described in Section 3.10 of the code [3]. Unlike the AASHTO Specifications, the CHBDC uses a general equation to determine wind load with factors that vary depending on the application of the load. The CHBDC Table A3.1.1 provides wind pressures for specific locations with wind pressure data for 10, 25, 50, and 100 year return periods and a recommendation in Cl. 3.10.1.2 suggesting that for roadside structures, the 50 year return period be used for overhead sign structures and 25 year return period be used for luminaires 16m or shorter [3]. On the contrary, the AASHTO Specifications have a wind load equation particular to roadside structures. Per AASHTO, the return period of 700 years is suggested to be used for traffic volumes of 100 < ADT ≤ 10000 when the risk category is typical (failure could cross travelway) and the 1700 year return period is suggested to be used for locations where the ADT > 10000 and the risk category is typical or whenever the risk category is high. AASHTO provides a series of contour maps of the United States for various MRIs that can be used determining the wind speed for design. A comparison of the wind pressure calculation from each specification is provided in **Tables 5** and **6** for Winnipeg, Manitoba.

Table 5: AASHTO Specifications (2015) Unfactored Wind Pressure Calculation

AASHTO Specification (2015)	Winnipeg, Manitoba
Equation 3.8.1-1	$P_z = 0.00256 K_z K_d G V^2 C_d$ (psf)
K_z (Table 3.5)	1.0
K_d (Table 3.8.5-1)	0.85
G (Clause 3.8.5)	1.14
V (1700 MRI) (Figure 3.8-2a)	120 MPH

AASHTO Specification (2015)	Winnipeg, Manitoba
C_d (Assumes Traffic Signals) (Table 3.8.7-1)	1.20
P_z = 42.9 psf (2.1 kPa)	

Table 6: CHBDC 2014 Unfactored Wind Pressure Calculation

Canadian Highway Bridge Design Code (S6)	Winnipeg, Manitoba
Equation 3.10.2.2 (Loading for horizontal support)	$F_h = qC_eC_gC_h$
q_{50} (Winnipeg) (Table A.3.1.1)	450 Pa
C_e (Table 3.9)	1.0
C_g (Clause 3.10.1.3)	2.5
C_h (Assumes octagonal member)	1.2
F_h = 27.2 psf (1.3 kPa)	

The loading calculated using both the AASHTO Specifications and the CHBDC are subjected to loading factors as prescribed in the respective standards in line with LRFD. However, the 2015 AASHTO Specifications only uses a factor 1.0 for wind loading for any given combination. The CHBDC on the other hand has factors ranging from 0 to 1.40. It is difficult to address the wind loading independently of the other applied loads and subsequent factors but if all else was equal, the 1.3 kPa calculated using the CHBDC would factored to 1.82 kPa.

For the equivalent unfactored pressure from the CHBDC equation to be calculated from the AASHTO equation, the wind speed would need to be 95 MPH; alternatively, the 50-year wind pressure would need to be 710 Pa for the loading calculated by the CHBDC to equal that of AASHTO's. There are no locations in Manitoba where the CHBDC specifies a pressure as high as 710 Pa be used for design and so the use of AASHTO is considered conservative in that province. There are however, regions in Canada where the pressure is significantly higher. The values in the CHBDC are more particular to the area of installation than the broad United States contour maps provided in the AASHTO Specifications. Depending on the area of interest, the value calculated by the CHBDC may be higher or lower than AASHTO's and diligence should be exercised before using a non-regional standard to ensure the differences are understood if the values vary from one to the other.

3.2 Fatigue Category Selection

The selection of the fatigue category is often the governing factor in design and as such warrants substantial consideration during selection of the category. Roadside structures are subjected to fatigue that if not properly considered, can lead to failure of the structure. An example of such a failure is presented in Figure 2.

Figure 2: Failure of Cantilevered Structure as a Result of Fatigue (Left) and Failure Location at Arm/Flange Connection (Right)

The figure shows a failure of a structure along a major route in Manitoba. It was suspected that the failure was a result of the fatigue damage incurred over the life of the structure.

One of the concerns with fatigue-induced stress is that even low amplitude vibration can cause performance issues if sustained over long periods of time, as is the case for roadside structures. There is however, a subjective nature to fatigue design in the AASHTO Specifications that provides the engineer some flexibility in the design. The AASHTO Specifications identifies overhead cantilevered and non-cantilevered sign structures, overhead cantilevered and non-cantilevered traffic signal structures, and high-mast lighting towers (HMLT) as all requiring fatigue design. The focus of this section will be on overhead sign support and traffic signal structure fatigue design across jurisdictions and the factors influencing the design process.

Table 7 presents the responses from the DOTs regarding what kind of considerations are made based on the historical performance of their structures.

Table 7: Past Performance Considerations for Fatigue Category Selection

Location of DOT	Does your DOT take into consideration past performance of structures when selecting a fatigue category?
Province of Alberta	No. The fatigue design information provided in AASHTO is not that old. We do not evaluate past structures when selecting fatigue category for design of new structures because most of the older structures were not designed to AASHTO fatigue criteria as it did not exist. No fatigue design information has been provided in the CHBDC either to this point.
Province of British Columbia	No response
Province of Manitoba	Yes. Past design performance has informed current design practice
Province of Nova Scotia	We follow The Canadian Highway Bridge Design Code (S6)
Colorado	Yes
Illinois	No response

Location of DOT	Does your DOT take into consideration past performance of structures when selecting a fatigue category?
Montana	Yes. Past performance is why we reduced the Fatigue Category for luminaires and signal structures
New Hampshire	Yes
Oregon	Yes
South Dakota	Yes
Utah	No

A majority of respondents are shown to consider past performance of structures to inform their current design practice. Interestingly, in some cases such as the DOT in Montana, past experience has led to less conservative design practice with a lower fatigue category used for luminaires and traffic signals. In Manitoba on the other hand, the lessons learned from existing structures have led to a more conservative fatigue category selection in some cases. A counter point was raised arguing that because the fatigue considerations in AASHTO have not been in practice for a significant period of time, the behaviour of existing structures is not yet a definitive example of the performance of the design for fatigue. In the case of Manitoba however, past performance was used to inform a decision on increasing the fatigue category selected to improve performance. The effect of different fatigue categories on design will be discussed further in Section 5. The DOTs were also asked to provide the fatigue category used for design. The responses are provided in **Table 8**.

Table 8: Fatigue Category used for Design

Location of DOT	Fatigue Category Selection
Province of Alberta	Category I used for all structures.
Province of British Columbia	No response.
Province of Manitoba	Category II for Medium Series Structures and below and Fatigue Category I for Heavy Series Structures and above with respect to signal structures and advanced warning structures.
Province of Nova Scotia	The CHBDC is followed.
Colorado	S-614-40 and S-614-40A designed with Category II excluding galloping. Other structures designed with Category I.
Illinois	Fatigue Category I for all OHSS. Galloping and truck induced gusts are not required unless specific structures have issues.
Montana	Fatigue Category I for sign structures. Category II for luminaires and signals.
New Hampshire	Selected from Table 11.6-1 in AASHTO Specifications
Oregon	Category I for all monotube cantilever structures
South Dakota	Category I
Utah	Category I for all situations

Fatigue Category I is the predominant fatigue category used across the DOTs with several using it exclusively. Three of the respondents explicitly use a combination of Fatigue Category I and II depending on the structure it is applied to. The DOTs that use a combination of fatigue categories (Manitoba, Colorado, and Montana) are among those that responded that past performance influences the decision of which fatigue category to use.

3.3 The Use of Mitigation Devices

Designing roadside structures for fatigue is important for safety as one of the common forms of structural failure, when it does occur, is from fatigue failure. Additionally, fatigue design often governs the design of the structure and therefore dictates the sectional properties of the structure. Because of the safety concerns and the economic considerations associated with more substantial sections, fatigue mitigating devices have been investigated through various research programs in recent years. The benefits of providing an effective dampening system are that the structure experiences a reduced vibrational amplitude, and by extension, a smaller stress induced by the vibration meaning the same structure with a dampening device is inherently safer in terms of potential for failure as a result of fatigue. Further, the fatigue requirements may be reduced which may result in a reduced section for the structural members possibly equating to a reduction in cost. While reports such as the NCHRP Report 412, 469, and 718 discuss mitigation devices, the widespread and substantive reduction in fatigue stress considered has been slow in developing as seen, in part, in **Table 9**.

Table 9: Reduction of Fatigue Category as a result of the Application of a Dampening System

Location of DOT	Have dampening or energy-absorbing devices been used to reduce Category I structures to Category II? If so, which products have been incorporated? Has their performance been documented?
Province of Alberta	No. There is so much variability in making dampeners or energy absorbing devices work with a structure. We do not allow dynamic analysis to be used in lieu of the static load design method due to the risk associated with modelling error/accuracy of the actual fabricated structure and its effect on dynamic behaviour
Province of British Columbia	No response
Province of Manitoba	Wind deflectors are used in some instances. The Fatigue Category is not reduced as a result
Province of Nova Scotia	No
Colorado	Yes, S-614-40 and S-614-40A has a special fatigue resistant design for the mast to pole connection. There has not been a need to install dampening devices
Illinois	We do use Stockbridge style dampers on all OHSS, but do not reduce to Fatigue Category II. Some documentation exists on their performance
Montana	No. Damping devices are attached by MDT if the structure shows severe galloping after installation
New Hampshire	No
Oregon	No. These can fail over the 50 year life span and result in no damping
South Dakota	No

Location of DOT	Have dampening or energy-absorbing devices been used to reduce Category I structures to Category II? If so, which products have been incorporated? Has their performance been documented?
Utah	No

In general, dampening devices are not used across the DOTs, and when they are, the fatigue category designed for is not reduced. Dampening devices appear to be viewed as contingency items that may improve the performance but are not seen as so reliable that the design can be altered as a result of the inclusion of the device; this is despite the 2015 AASHTO Specification's allowance that galloping fatigue can be neglected for cantilever and traffic signal structures if a device is included (Clause 11.7.1.1) [2]. NCHRP Report 469 Section 3.2.1.4 discusses the use of mitigation devices to reduce the importance Fatigue Category from I to II but does not negate fatigue loading entirely [4].

Continued research is needed in order to make dampening devices an accepted approach to safe and economic design. Research is needed in the area of service life of the device itself to convince designers that the device will be able to work effectively over the service life of the structure. Further research is needed to show that the dampening device is effective over a diverse range of environmental conditions and that the fatigue category may be safely reduced as a result of real-world behaviour of the structure. We are currently in in discussion to develop a research program to observe existing structures in the field with dampening devices installed compared to those without. Both sets of structures would have strain gages and/or accelerometers attached to monitor the structure over time in tandem with data acquisition systems to collect the data produced by the measurement device. The in-situ performance of the dampening device could then be compared to the structure(s) without the device to determine the long term performance of the device itself and the structure(s) where vibrations are intended to be mitigated.

Dampening devices are not the only way of attempting to prevent failure as a result of fatigue stress. Design details such as backup strips and the specific connection detail at the shaft-to-arm connection can increase the fatigue performance [5]. The design details used by the responding DOTs are discussed in Table 10.

Table 10: Use of Fatigue Mitigation Devices

Location of DOT	Are any other fatigue mitigation details used? (For example back-up strips)
Province of Alberta	No. We used to use post-tensioned dywidag anchor rods in the foundations to alleviate concerns with fatigue in the anchor rods. We no longer use this based on consultation with industry (design/fabrication) partners and their recommendations
Province of British Columbia	No
Province of Manitoba	Back-up strips are used at the base of the vertical shaft and at the shaft end of the cantilever arms. Gusset plates are used in the vertical shaft where the arm is installed for increased stiffness
Province of Nova Scotia	We have used FRP in the past as a rehabilitation measure

Location of DOT	Are any other fatigue mitigation details used? (For example back-up strips)
Colorado	Yes, S-614-40 and S-614-40A has a special fatigue resistant design for the mast to pole connection
Illinois	No response
Montana	No
New Hampshire	No
Oregon	Horizontal 2.5' x 3' sign blank mounted towards the end of the arm
South Dakota	No
Utah	No

Only Manitoba employs the use of backup strips in order to improve the stiffness of the structures and thereby improve fatigue resistance [5]. Nova Scotia has used fibre-reinforced wraps in the past in order to improve the stiffness as a rehabilitation measure after damage is incurred or is at risk of being incurred as evidenced by their response above. In general it is inferred that the DOTs are sufficiently satisfied with designing to Fatigue Category I in most instances and the installed structures have performed well enough for that to continue to be the case without the need for dampening devices or details that further improve the stiffness of the structure. The outstanding argument to be addressed is one of economic viability of the structures designed. This argument is addressed in Section 4.0.

4.0 Case Study of Significant Update to Design in Manitoba

Manitoba Infrastructure (MI) recently updated the design catalog for their traffic signal structures and advanced warning structures. The updates included a change from working stress design (WSD) to load and resistance factored design (LRFD) and updated importance category selection for fatigue considerations. The basis for the update was a desire for improved performance of the structures and a need to fill in gaps in structures that could be chosen for applications in the province and have a more user-friendly catalog that increased clarity in what combinations of signage could be used within the acceptable design envelope. To that end Dillon Consulting Limited (Dillon) proceeded to update MI's design using the AASHTO Specifications (2015). The following section analyzes the differences between the design considerations and final dimensions of the components for a ~6.8 meter tall shaft with a ~12.8 meter long cantilevered traffic signal structure using the design from before and after the change in design specifications. The case study will identify differences in the two structures and discuss the impacts of different design philosophies.

4.1 Comparison of Design Using the 2009 and 2015 AASHTO Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals

Previous sections have discussed the changes to the AASHTO Specifications over the previous versions. Manitoba has recently undergone an update to the design of the traffic signal structures and advanced warning structures in the province which provides an opportunity to analyze the impacts of updating the version and design considerations used. **Table 11** provides a comparison for two similar structures: one designed with the 2009 specifications and one with the 2015 version.

Table 11: Component Dimensions for 7.5m Cantilever Structure Designed with AASHTO Specifications

Traffic Signal Structure Component	AASHTO Specifications (2009)	AASHTO Specifications (2015)
	Dimension (mm) UNO	Dimension (mm) UNO
Total Shaft Height	6800	6800
Height to Center Line of Arm	6500	6500
Across the Flats at Base of Shaft	260	220
Across the Flats at Top of Shaft	260	220
Octagonal Shaft Thickness	7.94	9.53
Arm Length	7500	7500
Across the Flats of Arm at Shaft	165	220
Across the Flats of Arm at End	75	75
Octagonal Arm Thickness	6.35	9.53
Weight of shaft and arm steel (kg)	374	380

The 2009 structure in **Table 11** was design using Fatigue Category I for both natural wind gust and truck induced fatigue. Galloping was not considered, however.

The 2015 structure was designed using Fatigue Category II for natural wind gust and truck induced fatigue as well as galloping. Details were also updated such as including backup strips at the base of the shaft and at the arm at the shaft to increase stiffness at the connections. The Fatigue Category for the newly designed structure was chosen as Category II because of the lower travel speed on roads where those structures are likely to be present and lower potential impact of failure compared to larger structures. The DOTs in Colorado and Montana responded that they have chosen a similar tact to choosing the Fatigue Category with Montana using Fatigue Category II for luminaires and traffic signals and Fatigue Category I for OHSS. Colorado uses Fatigue Category II with no galloping for two particular design and Fatigue Category I for the remaining configurations.

The two structures compared above are similar in sectional properties with the structure designed based on the 2009 Specifications have a larger across-the-flats dimension for the shaft but lower thickness of steel compared to the structure designed with the 2015 Specification. The total weight of steel for the shaft of the initial structure is 374 kg compared to 380 kg for the newly designed structure. The same configuration would work for the new design if galloping were not considered. However, when galloping is included in the design the fatigue stress at the arm where it connects to the flange well exceeds the CAFL for that component. It was therefore necessary to increase the across the flats at the connection to the flange as well as increase the steel thickness. This larger section increased the load to the shaft which was subsequently considered.

Another area that affects the design of the structure is the number of signals and signage that is considered for each design. The attachment area changes the loading the structure is subjected to by increasing the surface area the wind pressure is applied to as well as incurring an increase in shear and flexure as a result of the weight of the attachments. The prior design of the 7.5m cantilever structure considered one signal on the arm at the end of the arm, a sign blade at 4.1m from the shaft on the arm, a pole signal 4.0m from the base, a pedestrian signal 3.3m from the base, and a pole sign cluster with $2m^2$ area. The updated design has signage shown in **Figure 3**. The amount of area on the arm of the updated design is slightly greater with a 2.6 m^2 cumulative area of attachments compared to 2.0m for the previous design.

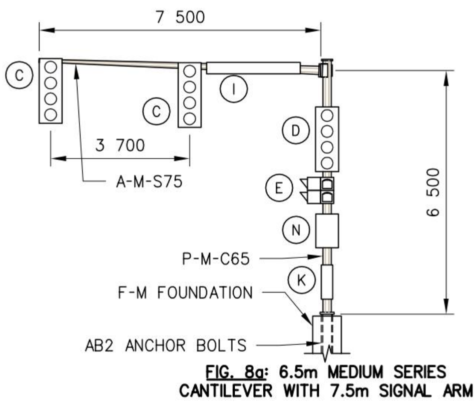


Figure 3: New 7.5m Cantilever Medium Series Traffic Signal Structure

Despite the higher Fatigue Category used in the previous version of the design, the inclusion of the galloping fatigue with the Category II Importance Factor applied provided the most severe fatigue pressure of any of those calculated. The galloping pressure was calculated as 0.60 kPa compared to 0.59 kPa for the truck induced gust value with Fatigue Category I in the previous design. The equations for galloping, natural wind gust, and truck induced fatigue from AASHTO are shown in Equations (1)-(3), respectively:

$$P_G = 21 I_F \ (psf) \ (1)$$

$$P_{NW} = 5.2 C_d I_F \ (psf) \ (2)$$

$$P_{TG} = 18.8 C_d I_F \ (psf) \ (3)$$

Typical values of the drag coefficient, C_d, are approximately 1.2. Further, the values for the importance factor, I_F, **Table 12**.

Table 12: Importance Factor Values for Cantilevered Traffic Signal Structures

	Fatigue Importance Factor, I_F		
Fatigue Category	Galloping	Natural Wind Gusts	Truck-Induced Gusts
I	1.0	1.0	1.0
II	0.65	0.80	0.85
III	0.30	0.55	0.70

As a result of the large constant in the equation for galloping fatigue of 21 the galloping fatigue stress commonly governs fatigue considerations in design, even when considered at a lower category than natural wind gusts and truck-induced gusts.

5.0 Case Study of the Application of Mitigation Devices

The use of mitigation devices have been discussed as a means to reduce the amplitude of vibration and potentially reduce the fatigue category selection chosen for design. Concerns remain as to the service life of the mitigation devices and the efficacy of the devices. Devices such as the Valmont Mitigator TR1 have however shown promising ability to reliably reduce vibration in cantilevered structures [6]. The TR1 has also undergone field testing where it has increased damping by an average of 1.02% over the four sites tested equating to an average reduced response of 89% [6].

Because the mitigation devices serve to limit fatigue stress which typically slowly causes catastrophic damage in structures, it is possible to have the device fail without the structure failing. If the device can be replaced or repaired before the structure has experienced an extended period of time in the undampened state, the resulting damage caused by larger fatigue stress may be reasonably mitigated. Research into structures with periods of damped and undamped behaviour and the service life of those structures would further aid the outlook on widespread use of the devices.

With the potential for devices to significantly reduce fatigue stress, it is prudent to discuss the repercussions that using the devices on structures might have. Section 3 discusses the NCHRP suggesting a reduction in Fatigue Category from I to II. The consequences for the design of a representative structure under each Fatigue Category I and II are subsequently investigated.

The same 7.5m cantilever structure as discussed in Section 5 is used for the comparison. The thickness of the sheet was set at 9.53mm to reduce the number of variables in the analysis. This comparison simply serves as an example of the effects a change in fatigue category can have on the material use and additional space needed for the footprint of the structure depending on the category selected. The comparison is by no means an exhaustive analysis of the effects of reducing the fatigue category.

Table 13: Component Dimensions for 7.5m Cantilever Structure Designed AASHTO Specifications (2015)

	Fatigue Category I	Fatigue Category II
Traffic Signal Structure Component	Dimension (mm) UNO	Dimension (mm) UNO
Total Shaft Height	6800	6800
Height to Center Line of Arm	6500	6500
Across the Flats at Base of Shaft	275	220
Across the Flats at Top of Shaft	275	220
Octagonal Shaft Thickness	9.53	9.53
Arm Length	7500	7500
Across the Flats of Arm at Shaft	250	220
Across the Flats of Arm at End	75	75
Octagonal Arm Thickness	9.53	9.53
Weight of shaft and arm steel (kg)	776	661

The amount of steel needed for the structure designed with Fatigue Category I compared to Fatigue Category II increases from 661 kg to 776 kg. This represents an increase of 17.3% without considering any necessary changes to fabrication costs or costs due to increased hardware costs such as flange or anchor bolts. There is also the potential for the structure to see an increased service life because of the reduced accumulation of damage from fatigue stress.

6.0 Conclusion

A review of both the CHBDC and AASHTO Specifications was carried out to look at the development of each standard and identify differences between the two. The method of design was discussed and in particular fatigue design was investigated to understand the changes over time. It was found that the CHBDC has made less progress in developing material for roadside structure design than the AASHTO Specifications. The AASHTO Specifications have progressed from defaulting to another standard for fatigue design to having a comprehensive fatigue section.

A survey was sent out to several DOTs to gather information or how practice is similar and different from region to region. In general, Canadian DOTs use CHBDC and American DOTs use the AASHTO Specifications. The one exception is Manitoba which uses the AASHTO Specifications. A majority of the respondents who do use the AASHTO Specifications have updated at least a portion of their designs to the 2015 AASHTO Specifications. The respondents were also asked to comment on fatigue design considerations. Three of the respondents noted that past performance of structures in the DOT is considered for current design practices. This has led to those three DOTs to have different Fatigue Categories for different structures whereas the other DOTs typically use Fatigue Category I for design.

Two case studies were also looked at – one for a recent update to designs of traffic signals and advanced warning structures in Manitoba as well as the effects of different Fatigue Categories on the cross-sections of structures as a result of reductions due to the use of mitigation devices. The design update in Manitoba highlighted the impact various fatigue considerations can have on design.

Further, while the contacted DOTs do not employ mitigation devices on a consistent basis and there remains skepticism as to the longevity and efficacy of the devices the economic advantages of a device allowing a reduction in Fatigue Category from I to II is significant as shown in the comparison of two 7.5m cantilever structures designed with Fatigue Category I and Fatigue Category II. Further research in this area is warranted in areas such as the service life of the devices and the performance of the structure and device over time. The efficacy of the mitigation devices in events of severe loading should also be investigated.

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