WARRIP

WESTERN AUSTRALIAN ROAD RESEARCH AND INNOVATION PROGRAM

Management of Long-lever Cantilever Sign Structures

Author: Hanson Ngo, Simon Orton, and Mahes Rajakaruna

Revision B

AN INITIATIVE BY:

The Government of Western Australia

mainroads WESTERN AUSTRALIA

arrb

Summary

This project supports Main Roads Western Australia (Main Roads) in the management of long-lever cantilever sign (CSS) structures. As part of Perth's new Smart Freeways project, long-lever (between 16 and 22 m) cantilever sign structures were constructed and installed along the northbound carriageway of the Kwinana Freeway. Due to limited stock of steel circular hollow sections in Australia, alternative materials were sourced that were produced to a different specification than the Australian Standard. While compliance testing undertaken during construction demonstrated conformance, there were some concerns raised regarding long-term fatigue life and durability of the structures. Over the last 2 decades, a number of long-lever cantilever sign structures have failed within Australia and internationally. There have been numerous causes for these failures which include issues with design and construction which led to failure mechanisms like fatigue. Recent studies indicate that higher stresses and a greater number of fatigue cycles than were previously anticipated are caused by wind and upward pressure from large vehicles passing beneath the structures. While there have been improvements in the Australian Standards and Main Roads Technical Standards based on the learnings from these past events, the overall safety and long-term performance have not been assessed/verified.

To date, Main Roads has maintained a program of inspection and maintenance with a lower frequency of inspections than other jurisdictions due to the young age profile of Main Roads' CSS structures, which ranges from 2000 to 2017. A large number of structures were constructed in 2009 and 2016. In general, inspection data is available, but not all records are immediately or accurately entered into the database with some time lag, especially with the impacts of COVID-19. Main Roads' S*ign Gantry Guidelines (Level 1 and Level 2 Inspections)* (Main Roads 2013a) are used with the tiered system of inspection with several response types depending on the severity of the inspection process detecting a defect.

Technical risks related to this class of assets are not fully understood or quantified, and both the literature review and pilot risk workshop established the need to further investigate technical risks in relation to fatigue and some CSS where there may be hidden defects. The incremental damage scenario where repeated 'normal' loading results in structural failure are dealt with in this report in addition to the conclusions and recommendations.

This project reviewed premature failure of similar structures from other jurisdictions over the last 2 decades and, as a result, suggests several opportunities to adjust the monitoring and governance system around managing CSS structures. The key project learnings were:

- Long-term fatigue in CSS structures is caused by wind-induced vibrations due to 4 wind loading phenomena including natural wind gusts, galloping, truck-induced wind gusts and vortex shedding.
- The uncertainty in current fatigue performance understanding requires additional inspection regimes to trigger maintenance and replacement strategies that may impact the expected life of the assets.
- Main Roads inspection levels for CSS are lower than some Australian counterparts and would require a review.
- One of the main causes of CSS failure in Australia and overseas is fatigue and short-term monitoring of CSS structures for fatigue defects is not an effective means of managing the risks of failure. This requires a longer-term commitment to understand the operating performance of CSS structures and longer-term due diligence checks on the quality of the controls used to manage existing CSS structures.
- Asset integrity risk can be measured and evaluated at a physical asset management control level and at a second line assurance level using 2 levels of risk assessment.
- The high level of uncertainty and the dynamic nature of CSS technical risks requires 2 levels of risk review to ensure risk controls are being reviewed regularly. Building 2 levels of governance is essential to ensure the risk communication is linked to the Main Roads risk operating model.

- Due to high levels of uncertainty, professional judgement is required to select the critical risk factors for prioritisation of inspection, testing, repairs and asset replacement. Over time, this will be improved as knowledge increases of the current CSS asset base.

The risk of apparent greatest concern occurs in the scenario where there is a catastrophic collapse of a CSS that does not coincide with a major storm event or similar. This is most likely an incremental damage scenario where repeated ‘normal’ loading results in structural failure. Developing an understanding of these events will be enhanced by capturing representative data, which is sparse both in Australia and internationally. This will help to address 2 current expertise limitations:

1. Quantification of the response of CSS structures to typical daily excitation and the corresponding effects that these have on typical structures.
2. The limit states design/assessment approach, which is typically focussed on the response to extreme events. While fatigue is one of the potential limit states, a meaningful knowledge of it requires an understanding of (1) above.

This project recommends a short and medium-term monitoring program over 3–5 years to demonstrate due diligence and is likely to address the above expertise limitations. Due to high levels of uncertainty, a higher level of focus on control effectiveness (checking and inspection) is required until asset base performance and, additionally, a 3–5 year testing program are completed. Once the knowledge base is increased and risk is reduced, the increased monitoring and defect response program may be reconsidered as a priority.

Acknowledgements

The project team would like to acknowledge the contribution of various Main Roads staff members who are directly involved in the management of Main Roads sign support structures in providing expert advice and technical input to the risk workshop which was conducted as part of this project.

ARRB – YOUR NATIONAL TRANSPORT RESEARCH ORGANISATION

ABN 68 004 620 651
National Transport Research Centre and Head Office: 80a Turner St, Port Melbourne, 3207 VIC, Australia
With offices in Brisbane, Sydney, Adelaide, Perth.
arrb.com.au

Contents

Tables

Figures

1 Introduction

1.1 Background

Main Roads Western Australia (Main Roads) currently manages a large number of sign support structures, of which 64 are classified as long-lever cantilever structures (with a cantilever length of 10 m or above), including 8 structures (with 16–22 m cantilever arms) having been constructed recently as part of the Smart Freeways – Kwinana Northbound Project in Perth.

Over the last 2 decades, a number of long-lever cantilever sign support (CSS) structures have failed within Australia and overseas. There have been numerous causes for these failures, which include issues with design and construction which led to failure mechanisms like fatigue, vibration and vortex shedding. Recent studies indicate that higher stresses and a greater number of fatigue cycles than were previously anticipated are caused by wind and upward pressure from large vehicles passing beneath the structures. While there have been improvements in the Australian Standards and Main Roads Technical Standards based on the learnings from these past events, the overall safety and long-term performance have not been assessed/verified.

Concern has been raised about the compliance of the structures constructed on the Smart Freeways – Kwinana Northbound Project to the relevant Australian Standards and Main Roads specifications. These structures were designed to AS 4100, assuming the steel to be AS/NZS 1163 Grade C350 L0 for the main columns and gantry beams. While sourcing the steel circular hollow sections (CHS) required for the columns and cantilever beams, it became apparent that there was limited stock available that comply with AS/NZS 1163. In particular, CHS larger than 610 mm available were manufactured to overseas standards. However, the Australasian Certification Authority for Reinforcing and Structural Steels (ACRS) certification could not be obtained for these CHS, and it was not possible to find a product certifying body that could certify the steel.

To manage the risk from the component compliance concerns and past performance of similar structures, a management strategy is needed to ensure the safety of the travelling public and mitigate potential risks for Main Roads and the WA Government. This report reviews and proposes a risk management framework within the Main Roads operating model and proposes additional work to finalise the recommended actions.

1.2 Project Aims and Objectives

The overall aim of this project was to establish a knowledge base with respect to the performance and safety risks associated with long-lever cantilever sign structures. This aim was achieved through collating current best practice in the management of this class of assets, along with the Main Roads context, to benchmark Main Roads practice against best practice and identify any possible gaps. This will assist Main Roads and other road agencies to develop a methodology to monitor and proactively manage the risks associated with these structures. In particular, the project aim was achieved through the following objectives:

- Review of relevant international and Australian practice for failure modes and common issues, as well as inspection, testing and/or monitoring and management practices of long-lever cantilever sign structures.
- Review of available records of standard design, inspection reports, maintenance, and rehabilitation of selected existing sign structures in Main Roads WA.
- Review of the monitoring scheme proposed for the Smart Freeways Project.
- Assessment of the potential risks and opportunities with appropriate consideration of:
  - business objectives
  - technical objectives
  - in-service performance
  - development and/or modification of risk management strategies.

- Trial of the use of Main Roads' internal risk management procedures to ascertain practicality for use in the risk-based methodology identified for the CSS structures.
- Identification of a risk-based methodology to assist in the management of structural risks identified for Main Roads CSS structures including
  - risk-based inspection regime
  - instrumentation and continuous monitoring
  - maintenance program (if required).
- Preparation of a preliminary scope and costing for future work outside the scope of this project including:
  - implementation of instrumentation and continuous monitoring
  - modification to Main Roads 'Sign gantry inspection guidelines'.
- Conduction of a cost-benefit analysis for the implementation of the project outcomes.
- Documentation of the findings in a WARRIP report.

1.3 Project Scope

The project scope includes:

- relevant publications related to the relevant international and Australian practice for common issues as well as inspection, testing and/or monitoring and management practices of long-lever cantilever sign structures
- Main Roads available database of long-lever cantilever sign support structures
- Main Roads relevant documentation which was made available during the delivery of the project.

The following were excluded from the scope of this project:

- sign support structures other than long-lever cantilever structures on highways
- field work.

1.4 Project Outline and Framework

In addition to this Introduction section, the report includes the following sections:

- Section 2 reports the outcome of a literature review of the relevant international and Australian practice for common issues as well as inspection, testing and/or monitoring and management practices of long-lever cantilever sign structures.
- Section 3 provides a summary of a review of the current practice in Main Roads in the management of CSS structures and benchmarks its practice with the current best practice.
- Section 4 presents the finding of a risk workshop with technical input from Main Roads relevant staff who are directly involved in the management of the CSS structures in WA. A management framework was proposed in this section for Main Roads to manage the potential risks associated with these structures.
- Section 5 provides lessons learnt from the delivery of this project and recommendations for Main Roads going forward in the management of the CSS structures.
- Details of the outcomes of the risk workshop are included in Appendix B.
- A project concept is presented in Appendix C for monitoring of CSS structures.

2 Literature Review

2.1 Literature Search

A literature search was conducted at the Australian Road Research Board (ARRB)'s library. The purpose of the literature search was to source all available publications related to the relevant international and Australian practice for common issues as well as inspection, testing and/or monitoring and management practices of long-lever cantilever sign structures. The key words used in the search included 'Cantilever sign structure' in combination with the following words:

- fatigue
- monitoring
- testing
- vibration
- wind effects
- failure mechanism/mode
- issues
- defects
- vortex shedding
- inspection
- condition assessment
- strengthening
- material compliance
- risks
- management strategy.

The literature has been sourced from the following databases, in addition to the use of general search engines:

- Australian Transport Index (ATRI) – an extensive database of information on land transport publications which contains over 180,000 records
- TRID (Transport Research International Documentation) – an integrated database that combines the records from the US Transportation Research Board (TRB)'s Transportation Research Information Services (TRIS) database and the Organisation for Economic Co-operation and Development (OECD)'s Joint Transport Research Centre's International Transport Research Documentation (ITRD) database. TRID provides access to more than 1.25 million records of transportation research worldwide.

Based on the literature obtained, the most relevant publications were selected for detailed review.

2.2 Current Practice – Australia

2.2.1 Main Roads

Main Roads carries out Level 1 (Level 1) inspections annually or after an event occurs that may impact the condition of the structure. Level 2 (Level 2) inspections are carried out every 7 years, which include visual inspection, weld inspection, measurement of steel thickness and protective coatings. Special inspection and investigation (Level 3) are carried out when necessary.

A checklist for what to look for on each component is provided in S*ign Gantry Guidelines (Level 1 and Level 2 Inspections)* (Main Roads 2013a). Four condition states are provided with detailed descriptions for each component.

There are no failure events of cantilever sign structures in WA.

Refer to Section 3 for more details.

2.2.2 Queensland Department of Transport and Main Roads (TMR)

TMR currently manages a number of CSS structures on its network. A maximum span for CSS structures of 9.5 m (from centreline of the column to the end of the horizontal arm) is preferable, while spans greater than 9.5 m can only be adopted with the prior approval of the Director (TMR 2021). Design for fatigue is the focus of the design criteria and is specified to be in accordance with the latest revision of the AASHTO *LRFD Specification for Structural Supports for Highway Signs, Luminaries and Traffic Signals* (AASHTO 2015). CSS structures shall be designed in accordance with TMR Standard Drawing 1581 *ITS Cantilever: Cantilever Structure*.

The following modifications to the design of CSS structures in Queensland have been made based on learnings from the US practice and interstate sign failures:

- Connections using Class 4.6 bolts are less likely to be controlled by fatigue requirements than those which use Class 8.8. Either grade may be used, however, provided that the design satisfies both ultimate strength and fatigue life requirements.
- Pre-tensioned bolts are not warranted.
- Nut tightening and the use of lock nuts are important to prevent nut loosening, which in turn will cause undesirable larger stress fluctuations, thus increasing the risk of fatigue failure.
- Snug tightening is adequate for fatigue performance provided it is done properly and provided lock nuts are:
  - purpose made half height nuts
  - made from softer steel than the ordinary nut, and
  - located between the ordinary nut and the base plate.
- For Class 8.8 bolts, to prevent loosening more reliably, tightening in the range from 1/2 turn to 1/3 turn beyond snug tight, depending on bolt class and diameter, is now preferred (in conjunction with lock nuts).
- The maximum unsupported length of the anchor from the top of footing to the bottom of the base plate is to be not more than one anchor diameter.

Guidelines for inventory and inspection of large traffic management signs are undertaken in accordance with the *Structures Inspection Manual* (TMR 2020). Key areas for inspection include:

- missing, loose or damaged nuts/bolts
- cracked welds
- butt welds at structural connections
- corrosion
- splits or ruptures in columns and stiffeners
- impact damage
- tilting columns
- crushed/missing mortar beneath base plates
- exposed levelling nuts beneath base plate (levelling nuts should not be engaged in carrying load after mortar has been placed)

- base plates or other steelwork which has been incorrectly (i.e. not in accordance with the drawings) encased in grout or concrete (any affected components will need to be exposed to determine the extent of any corrosion)
- cracking/spalling of concrete around base plates.

Details of what to look for in the inspection of each component group are provided in Table 2.1. Four condition states (CSs) are used for components and 5 used for overall structure. Detailed condition descriptions are provided for 4 condition states of each component depending on the severity and extent of defects.

Table 2.1: Inspection items of TMR's sign support structures

Components	Defects
Footings	• obvious evidence of spalling, cracking or reinforcement corrosion • rotation/settlement • erosion/undermining
Base plates, fittings and hold-down bolts	• cracking, spalling or voids in mortar pad • debris/fill over base plate • corrosion of fixings/base plate • loose/missing fixings and thread engagement
Columns	• corrosion, buckling, bending, rupture, rotation or misalignment of sections • impact damage • verticality of members • protective coating loss • loose/missing fixings and thread engagement
Cantilever arms/gantry beams	• corrosion, buckling, bending, rupture, rotation or misalignment of sections • separation/distortion at joints/splices • sagging of members • protective coating loss • loose/missing fixings and thread engagement
Signage and ancillaries	• completeness, damage, cleanliness, orientation and visibility to road users

Source: TMR (2020).

Similar to bridges and culverts, 2 levels of regular inspections are undertaken for CSS structures, including Level 1 and Level 2 inspections (visual inspections). Level 3 inspections are in-depth inspections and are carried out when necessary to provide a detailed assessment of defects or following repair activities, vehicle collision or natural disasters. The inspection frequency is dependent on overall condition state: Level 1 inspection: annually; Level 2 inspection: 2 years for CS1–2, and 1 year for CS3–4. Level 1 and Level 2 inspections are staggered by 6 months to ensure inspections occur every 6 months for CS3–4.

TMR implements the same maintenance, repair, rehabilitation, and management approaches for CSS structures as for bridges. The following information regarding sign support structures managed by TMR is based on personal conversations with several TMR personnel:

- Gantries are currently managed by Districts.
- There are not a lot of CSS in Qld.
- No significant gantry failures have been reported in recent times, even after large storm/flooding events. This is possibly due to the fact that TMR is currently implementing a robust, high frequency inspection regime; therefore, maintenance, repair or replacement work shall be taken in a timely manner. Consequently, no structure has been left in poor condition that could result in a collapse.
- Currently a review of asset management practice for gantries is in progress which is planned to be completed by end of the year.

- The database of the CSS structures is available; however, the level of accuracy cannot be verified before the current review is completed.

2.2.3 Department of Transport Victoria (DoT)

Following a few fatigue-induced failures of CSS structures in Victoria in the past (refer to Morris & Thomas 2009), DoT (formerly VicRoads) has revised its technical specification to mitigate the risk of these failures. The following improvements have been specified based on learnings from past failures (VicRoads 2019a):

- Fatigue design shall be in accordance with the current version of AASHTO *LRFD Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals* (AASHTO 2015).
- Fatigue design loads for sign gantries, including drag coefficients, are calculated for an infinite fatigue life, Fatigue Category I in accordance with the AASHTO specifications.
- A number of amendments have been made to AS/NZS 5100.6:2017 regarding the requirements for connection details to avoid conditions that create highly constrained joints and crack-like geometric discontinuities that are susceptible to constraint-induced fracture. In addition, finite element analyses are not accepted on their own as adequate to determine fatigue strength without being backed up by rigorous physical testing carried out in accordance with international standards.
- The potential for a resonant response of the cantilever arm of the cantilever sign structures to vortex shedding originating from the column shall be assessed, including designs in which steel box sections are used for the principal members. If a resonant response is possible, this shall be mitigated in the design (for example by the installation of impact dampers or wind-flow spoilers).

Risk management

In addition to improving the fatigue design practice, DoT also pays great attention to the risk management of CSS structures to mitigate the potential risk of collapse and additional risk to third party road users. Guidelines for protection of gantry and cantilever sign supports located in close proximity to traffic lanes are provided in Road Design Note 06-13 (VicRoads 2019b), in which barrier combinations are considered to provide both forgiving and higher containment barriers to reduce the likelihood of a vehicle occupant injury and structure collapse.

The following additional risks specific to gantry and CSS structures should include:

- consequence of a collapse, considering factors including the number of trafficked lanes affected by a collapse; probability that a vehicle is present during a collapse (e.g., traffic volume); and speed, sight lines and stopping distance provided for approaching vehicles
- provision of a safe workplace, considering need of a higher barrier containment level based on the frequency of maintenance, the routine tasks undertaken and the program to coordinate additional safety controls during work hours
- potential disruption to the network due to a collapse, considering factors including the number of trafficked lanes affected by a collapse, potential to detour traffic, direct and indirect network disruption, and removal and replacement disruption
- asset value/repair cost, considering the value of the asset over its lifetime and potential repair costs in the event of an impact.

Four risk categories (1–4) have been provided to classify the CSS structures, each with a minimum risk scenario and barrier criteria to assist in quantifying the additional risk for typical gantry and CSS structures. The required protection should meet the minimum barrier criteria provided.

Inspection types and frequencies

The inspection approach of the CSS structures is similar to that of other road structures such as bridges and culverts, in accordance with DoT's *Road Structures Inspection Manual* (VicRoads 2018). Four condition states are used to rate the condition of each component. Level 1 inspections are carried out every 6 months or after an event, while the frequency of Level 2 inspections is determined based on the specific structure's bridge condition number as shown in Table 2.2.

Table 2.2: DoT Victoria's Level 2 inspection frequency

Structure condition rating	Inspection frequency (years)
< 30	5
30–60	3
> 60	2

In addition to the above inspection regime, in order to ensure that cantilever arms of length exceeding 9 m will have an adequate fatigue performance, vibration monitoring is recommended for a period of 12 months after construction of a CSS structure. As a minimum, vibration monitoring shall be achieved by measuring variations in strain at the base-plate weld in order to establish that the strain range is less than the relevant constant-amplitude fatigue limit defined in AASHTO (2015).

2.2.4 Transport for New South Wales (TfNSW)

In NSW, 18–19 m cantilever arms have been constructed. Only one failure of CSS structures has been reported, which occurred near Blacktown in 2008 (refer to Section 2.3.3). While CSS structures constructed prior to 2009 were not designed for fatigue, new structures currently require the application of AASHTO (2015) in fatigue design.

Four categories of inspection, Levels 1–4 are specified for the assessment of structural integrity of CSS structures (Roads and Maritime Services (RMS) 2013), in addition to the initial inspection which is carried out shortly after the construction of the structure. Details of these inspection categories are as follows:

- Level 1: includes non-scheduled discoveries by the public or road crew/road surveillance.
- Level 2: are scheduled structural integrity rating inspections that are carried out in accordance with the RMS (2013) manual by inspectors with appropriate training.
- Level 3: are Structural Engineering inspections/investigations carried out to respond to a Level 2 inspection report with adverse findings. Non-destructive testing techniques may be required. These inspections are undertaken by an experienced Structural Engineer.
- Level 4: are Structural Engineering inspections/investigations carried out by an experienced Structural Engineer for capacity assessment due to changes in loads associated with the proposed configuration of a new Sign and or Traffic Signal.

Non-destructive test methods including Ultrasonic tests, Dye Penetrant Tests and Magnetic Particle Tests are used as applicable in Level 2–4 inspections for the detection of corrosion and fracture of anchor rods, wall thickness loss in hollow sections, weld cracking and surface cracks.

The frequency of Level 2 inspections is determined based on the type of the structure. For example, for all steel structures and significant cantilever structures, a frequency of 5 years is applied; for cantilever structures with 4-rod anchorage systems of grade 8.8 or higher, the frequency is 2 years.

Five condition states are used for condition rating of structural elements, including Good, Satisfactory, Poor, Unsatisfactory and Critical. Feasible actions may be taken depending on the condition state. For example, the repair or replacement of an element in Poor condition within a specified timeframe or scheduling the repair or replacement of an element in Unsatisfactory condition to be undertaken promptly.

It should be noted that the TfNSW's (formally Roads and Maritime Services) *Processes and Procedures Manual for Structural Integrity Inspection and Condition Assessment of Traffic Asset Structures* (Roads and Maritime Services 2013) is currently under a major revision. One of the major changes is that the categorisation of sign structures will be risk-based, in which the risks are determined based on (i) consequences of failure e.g., loss of life, property and organisation reputation, and (ii) structural configuration, e.g., some structure types are more prone to failure than others in nature. In addition, the inspection frequencies are determined by the service provider in accordance with the guideline.

2.2.5 Department for Infrastructure and Transport South Australia (DIT)

In South Australia, inspection of CSS structures is primarily devoted to the structural condition aspects of the gantry, foundations, fittings, and the sign with its attachments (Department of Planning, Transport and Infrastructure (DPTI) 2020). Seven component groups are specified, including footings, masts, cantilever arms, hold-down bolts and fittings, base plates and fittings, sign face support structures, and ancillary components. Five condition states are used to rate the condition of each component.

DIT (formally DPTI) carries out regular Level 1 and Level 2 inspections for CSS structures. Level 1 inspections are undertaken at routine maintenance, the frequency of Level 2 inspections is determined based on age, overall length, span length and environment. A frequency of 4 years is applied to structures that are more than 40 years old, have an overall length of 50 m or higher, or have a span of 35 m or higher, or are located within 1 km of the ocean. For all other cases, a frequency of 8 years is applicable.

Level 3 – special inspections and investigations, are undertaken in a number of circumstances such as to respond to an incident, accident or natural event; assess the load capacity assessment of structures; or evaluate observations from Level 2 inspections or other condition data that is not visually evident.

Structures that show accelerated deterioration or are nearing the end of their serviceable life require an amended management process which will involve more frequent inspections. Monitoring inspections are carried out periodically and are visual, non-destructive inspections of specific components aimed at detection of structural distress that could indicate reduced strength.

2.2.6 New Zealand Transport Agency (NZTA)

In New Zealand, large traffic signs are defined (NZTA 2020) as those signs supported on 2 or more supports and installed on supports greater than 2.1 m apart. NZTA appears to have not experienced issues related to long-lever cantilever sign structures.

2.3 Current Practice – International

This section summarises findings from a review of a number of publications sourced from the various databases with the keywords listed in Section 2.1.

2.3.1 Types of Cantilever Sign Support Structures

Cantilever sign support structures are used extensively on major highways and at local intersections for traffic control and roadway illumination. The CSS structures are typically supported by a single vertical support rather than 2 or more supports for a traditional overhead support structure. The single support is usually located at the side of roads, which increases motorist safety by reducing the likelihood of vehicle collision. This structural form is attractive due to its lightweight, hence savings in material use; however, the flexibility of these structures is significantly greater than the multi-post overhead sign structures.

The length of large cantilever arm ranges from 9.5 m to more than 20 m; in some cases, CSS structures of a greater length have been constructed, such as 27 m arms constructed in Florida, USA (Kaczinski et al. 1998). The length of the cantilever arms has increased due to the increased setback for safety reasons and the construction of wider roads.

Materials

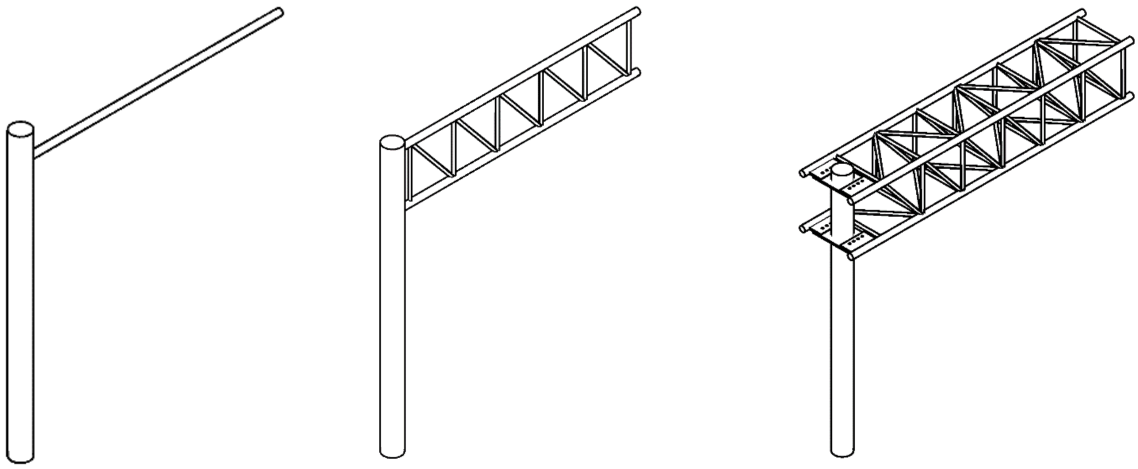
Reinforced concrete is usually used for the foundation of CSS structures, while painted mild steel, galvanised steel, stainless steel, and aluminium are commonly used for the poles and arms of the CSS structure's components, as follows (Kaczinski et al. 1998):

- painted mild steel: poles, mast arms, chords, and base plates
- galvanised steel: poles, mast arms, chords, trussing, base plates, hangers, hanger u-bolts, bolted structural connections, anchor bolts, and miscellaneous fasteners
- stainless steel: sign panel to windbeam fasteners, windbeam to hanger fasteners, pole to transformer base bolts, and miscellaneous fasteners
- aluminium: sign panels, backing strips, windbeams, and occasionally anchor bolt top nuts.

Types of cantilever arms

Cantilever arms may have the form of single chord, 2 chord plane truss, and 4 chord space truss (Figure 2.1).

Figure 2.1: Types of cantilever arms



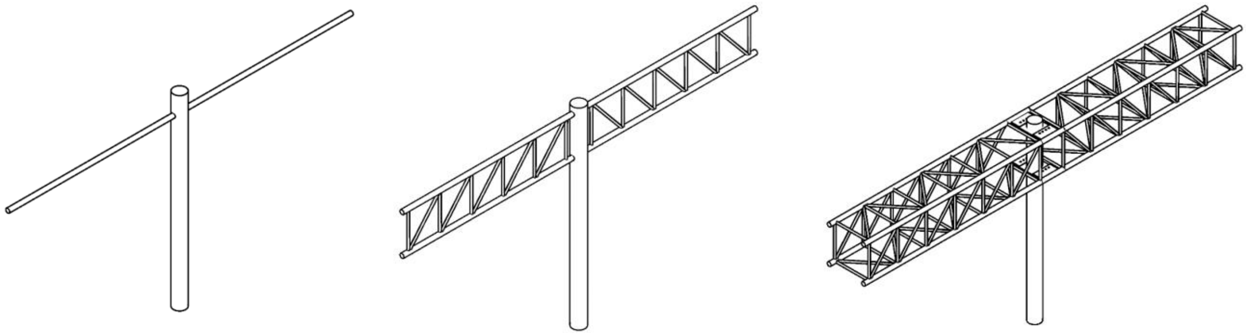
a. Single chord cantilever b. Two chord plane truss c. Four chord space truss

Source: Virginia Department of Transportation (VDoT) (2014).

- Single chord (Figure 2.1-a): consists of a single horizontal chord member which is typically of a tubular shape. The arm spans the roadway and supports the sign panels. This chord is supported at one end by a single vertical pole support.
- Two chord plane truss (Figure 2.1-b): consists of 2 horizontal chord members, with or without vertical and diagonal bracing members between them, to form a plane truss. The members of the truss are typically constructed of tubular or angular shapes. The truss is supported at one end by a single vertical pole support.
- Four chord space truss (Figure 2.1-c): consists of 4 horizontal chord members, with vertical, horizontal, and diagonal bracing members between them. The truss members are typically constructed of tubular or angular shapes. The truss is also supported at one end by a single vertical pole support.

Butterfly forms, as shown in Figure 2.2, have also been used for these structures.

Figure 2.2: CSS structures with butterfly arms



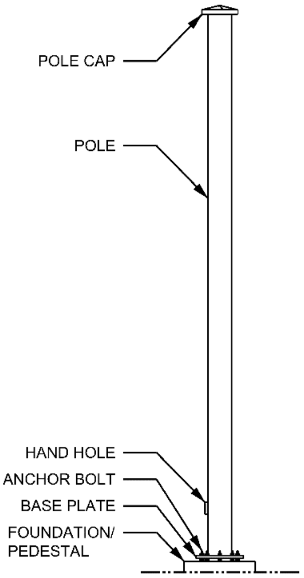
a. Single chord cantilever b. Two chord plane truss c. Four chord space truss

Source: VDoT (2014).

Pole types

The vertical supports of CSS structures most commonly consist of a single pole of a tubular or multi-sided shape. Associated components include pole cap, anchor bolts, base plate and pedestal/foundation (Figure 2.3).

Figure 2.3: Pole type and associated components



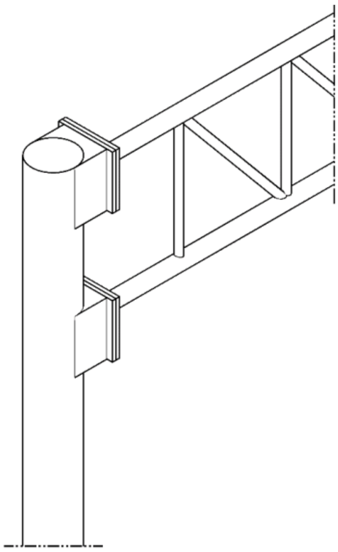
Source: VDoT (2014).

Connection between pole and arm

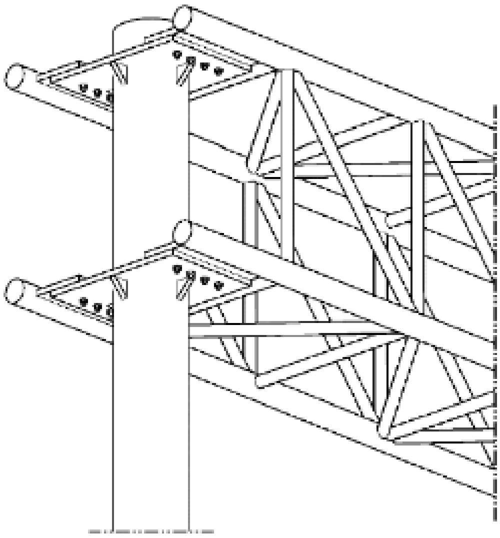
The cantilever arms of CSS structures are formed by joining multiple chord or truss sections. For tubular chords, splice flanges are welded to the chords at the end of each of the sections to form field bolt connections between truss sections. For angle chords, bolt or weld connections may be used to join the chord sections together, using a short section of angle that overlaps each chord section.

Depending on the number of chords, the connection between the arm and the pole may have differing connection details. Some of the connection types include gusseted box flange plates (Figure 2.4-a) or collar plates (Figure 2.4-b)

Figure 2.4: Typical chord to pole connection



a. Gusseted box flange plate connection



b. Collar plate type connection

Source: VDoT (2014).

Foundation

The foundation is the portion of the structure below the base plate, including the anchor bolts that attaches the pole, and transmits the structure's loads to the ground or substructure. Reinforced concrete spread footing is the most common for CSS structures; however, other types of foundation can be found in practice such as caissons or piles.

Other components include sign panels of various types (zee bar, extruded, hanging, luminaire, variable message sign) and walkways.

2.3.2 Load Effects on CSS Structures

The loads acting on the CSS structures include the dead loads of the structure's components and attachments (signs and accessories), and wind load effects. While the former are not significant and are usually straightforward to account for in conventional structural analysis, the latter are the governing load effects and are not easily quantified by theoretical approaches.

CSS structures are susceptible to various wind loading phenomena including galloping, natural wind, truck-induced gusts, and vortex shedding (Garlich & Thorkildsen 2005). With the flexibility due to long span lengths and small cross-sectional areas combined with low mass, these structures have low resonant frequencies of about 1 Hz, which is in the range of typical wind gust frequencies, and as a result, causes resonance or dynamic amplification of the response. In addition, the damping of the structure is extremely low, typically less than one per cent of the critical damping. With these dynamic characteristics, wind loads may cause large-amplitude vibration and/or fatigue cracking in the CSS structure's components (Dexter & Ricker 2002). Usually, the greater the length of the cantilevered mast arm, the more susceptible the support structure will be to wind-induced vibration.

Other factors contributed to the susceptibility of CSS structures to wind load effects include (Dexter & Ricker 2002):

- The advent of backplates (used on signal fixtures to block the sun and enhance the visibility of the signal) has increased the susceptibility to galloping of the signal support structures.
- The size and location of flat panel signs have also changed over the years – larger signs are now placed asymmetrically with their centre of gravity above the centre of gravity of the horizontal mast arm or support truss, increasing the torsional motion of the mast arms.

- Large variable-message signs (VMSs) are increasingly used, and these signs present a large horizontal surface that increases the effect of truck-induced gusts.

Details of the wind loading phenomena are briefly discussed in the following sections.

Natural wind gusts

The most common type of wind-induced vibration in sign support structures is from ordinary wind gusts or fluctuations of the wind velocity (Garlich & Thorkildsen 2005), which causes the horizontal deformation of the cantilever structure. Variable and randomly distributed structural response in the structural components are caused by varying wind impulses generated from the fluctuations in wind velocity with a wide range of frequencies. As a result, the possibility of resonance and aggressive vibrations is increased (Dexter & Ricker 2002). Cracks may initiate from the resulting large vibrations. For cantilever sign structures, cracks due to natural wind gusts usually develop over a period of at least several years and occur at the following locations:

- along the sides of the connection of the mast arm to the pole
- truss connections
- base of the pole
- weld joint between the pole and the base plate
- top of the stiffeners
- hand holes
- anchor rods.

The US experience was that CSS structures are particularly susceptible to vibration and fatigue from natural wind gusts in windy areas where the mean annual wind velocity is greater than 5 m/s. Fatigue cracking due to natural wind gust loading is more likely to occur in the areas where there are frequent constant winds than in the areas with the maximum peak wind velocities that are identified on wind maps. In addition, wind gusts caused by hurricanes or other large storms have little influence on fatigue failures of CSS structures.

The effects of natural wind gusts can be mitigated by altering the damping of the structure to lower the dynamic amplification factor (Kaczinski et al. 1998). This can be achieved by installing external damping devices. Increasing structural stiffness does not reduce the effects of natural wind gusts.

Truck-induced wind gusts

When trucks travel beneath sign support structures, horizontal and vertical gust loads are induced on the structure. The resulting horizontal load effect is negligible in comparison to that caused by the natural wind gust and is therefore usually ignored (Garlich & Thorkildsen 2005, Gallow et al. 2015). The vertical truck gust load generates a pressure load on the underside of the mast arm or truss and any attachments. As the truck-induced wind gusts have the same direction as the first 2 modal shapes of CSS structures (Hosch et al. 2017), the resulting pressure load creates an excitation with a magnitude and the same direction as the 2 modal shapes. Consequently, resonance may occur, causing large vibration of the structure.

Fatigue cracking from truck-induced wind gusts usually develops over a period of several years, but it has been reported (Garlich & Thorkildsen 2005) that a cantilever structure has failed when the structure was less than one year old. Similar to the natural wind gust effects, the cracks are usually located at the connection of the mast arm to the pole, at truss connections, at the weld joining the pole to the base plate, at the top of the stiffeners, at hand holes, or at the anchor rods.

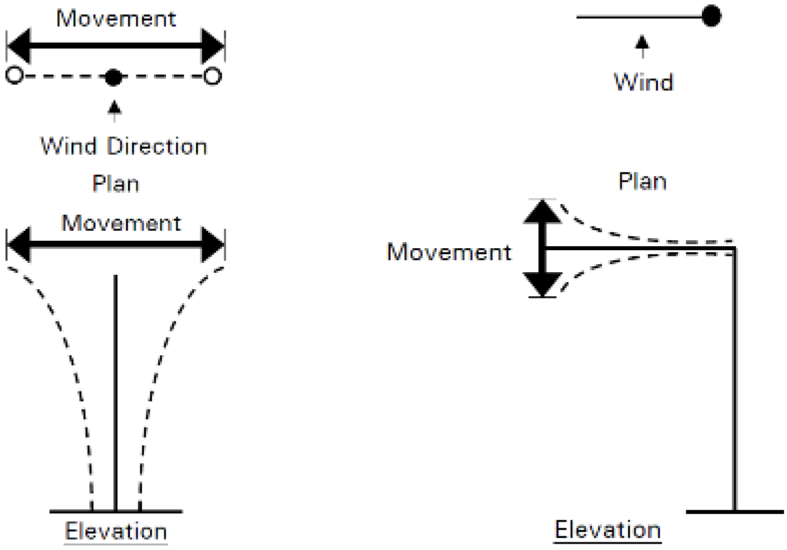
The truck-induced wind gust pressure is proportional to the speed of the trucks, so signs located on major highways and freeways are more susceptible than local roads where the trucks are traveling at lower speeds. In addition, the vertical truck-induced wind gust pressure varies in height; therefore, the greater the clearance between the tops of trucks and the bottoms of the signs, the less the susceptibility to truck-induced vibration. The truck-induced wind gust loads essentially reduce to zero at a height of 10 m above the roadway.

There are not many options to effectively mitigate oscillations of horizontal mast arms due to truck-induced gust loads (Kaczinski et al. 1998). It is possible, however, to reduce the magnitude of vertical truck gust pressures on the bottom of the sign by increasing the vertical clearance of the sign panel above the tops of trucks. This option may be limited by roadway geometric constraints and sight distance requirements. Additionally, the use of perforated panels and open grating can be used to reduce the effective horizontally projected area.

Vortex shedding

Vortex shedding is a phenomenon that occurs when the wind blows across a structural member, vortices are shed alternately from one side to the other, and where alternating low-pressure zones are generated on the downwind side of the structure giving rise to a fluctuating force acting at right angles to the wind direction (Garlich & Thorkildsen 2005). Vortex shedding can result in resonant oscillations of a pole in a plane normal to the direction of wind flow (Figure 2.5-a). The periodic frequency of the vortex shedding can resonate with the natural frequency of the pole, resulting in very large alternating forces acting transversely to the wind flow direction. Fatigue cracks can appear if the vibration is significant.

Figure 2.5: Vortex shedding and galloping effects on cantilever sign structures



a. Vortex shedding b. Galloping

Source: Garlich and Thorkildsen (2005).

It was reported by Garlich and Thorkildsen (2005) that vortex shedding tends to occur with steady continuous winds at a critical velocity which is in the range of 5–15 m/s. Significant vibration does not occur if the wind velocity is less than 5 m/s, while the wind with a velocity of greater than 15 m/s is generally too turbulent for vortex shedding to occur. Vortex shedding can 'lock-in' (i.e., when vortex shedding frequency becomes coupled to the natural frequency of the structure) and continue as the velocity increases or decreases slightly; however, the vortex shedding will stop if the velocity changes by more than 20 per cent. Gusty variable winds, such as might occur in a severe storm typically will not cause vortex shedding.

Contrary to some statements in the 2013 Specifications (AASHTO 2013), recent studies have verified that vortex shedding can occur in tapered as well as prismatic circular poles with almost any diameter. In a few cases, sign support structures have exhibited vortex shedding on the mast arms before the installation of attachments (signs or signal heads). After attachments have been installed, vortex shedding is no longer a problem. Consequently, Garlich and Thorkildsen (2005) recommended that mast arms never be erected before these attachments are mounted.

It was also reported that CSS structures were generally not susceptible to significant vibrations due to the shedding of vortices from the supports (Kaczinski et al. 1998). The current AASHTO specification (AASHTO 2015) indicates that vortex shedding is 'almost never' a significant action observed in failures in these structures. In contrast with the above perception, however, Morris and Thomas (2009) pointed out, through a field test, that there appears to be a link between vortex shedding from the vertical column and lateral vibration of the cantilever tip when the wind direction is parallel to the sign face on a particular cantilever gantry. Therefore, effects of vortex shedding should not be ruled out in the design of CSS structures.

Vortex shedding depends on a theoretically calculated critical wind velocity, which is proportional to the averaged measured natural frequency corresponding to the first vertical mode of vibration (Kaczinski et al. 1998). Effects of vortex shedding can be mitigated by ensuring that the critical wind velocity that causes lock-in is outside the wind velocity range within which vortex shedding is likely to occur (5–15 m/s). This can be achieved by either altering the dynamic properties of the structure (i.e., mass, stiffness, and/or damping), or altering the aerodynamic characteristics of the structure (i.e., use of strakes, plates).

Galloping

Galloping usually occurs to CSS structures subjected to periodically varying angles of wind flow, causing horizontal components to vibrate vertically (Figure 2.5-b). Similar to vortex shedding, galloping is also caused by uniform steady winds, rather than gusty winds, but over a larger range of wind velocity. In addition, galloping results in large-amplitude, resonant oscillations perpendicular to the direction of wind flow, however, it occurs to asymmetric elements (Kaczinski et al. 1998). Therefore, galloping effects occur on the mast arms rather than the poles. The mode of vibration for galloping is swaying of the mast arm in the vertical direction, as well as twisting. A large portion of the vibration and fatigue problems that have been investigated for CSS structures were caused by galloping. Galloping may be the most critical wind loading phenomenon that causes major fatigue deterioration for single-mast arm cantilevers (Ghaedi et al. 2016).

While flexible monotube CSS structures are particularly susceptible to galloping, it is not an issue for 4-chord and 3-chord horizontal supports (Dexter & Ricker 2002). Typically, the longer the cantilever mast arm, the more susceptible to galloping of the structure.

Sign attachments can also contribute to the galloping effects. The number of signs and/or signal heads, their configuration, area, connection detail, and the direction of wind flow significantly influence the susceptibility for galloping. While all types of signal heads and signs have been observed to be affected by galloping, signal attachments configured with backplates and subjected to flow from the rear are most susceptible to galloping (Kaczinski et al. 1998).

The fatigue cracking caused by galloping usually develops over a long period of a year or more where there may be many days of winds that cause galloping. In CSS structures, the cracks will usually occur at the connection of the mast arm to the pole, at the base of the pole, top of stiffeners, perimeter of hand holes and anchor rods.

Galloping-induced oscillations in CSS structures can be mitigated by altering the dynamic characteristics of the structure (mass, stiffness, damping) so that the magnitude of the onset wind velocity is greater than the velocity for which steady-state flows are typically maintained (Kaczinski et al. 1998). Alternatively, changing the aerodynamic characteristics of the attachments could also help, such as altering the geometry or installing damping plates.

The galloping loads are so significant that they typically govern the fatigue design when they are applicable. Therefore, mitigation devices will have significant cost benefits in reducing the effects of galloping as opposed to natural wind gusts or truck-induced wind gusts.

2.3.3 Common Issues

Common defects in the CSS structures can be classified as either durability issues or structural issues. The former is usually caused by environmental impacts and material-related deteriorations, the latter can be caused by a whole range of factors including fabrication imperfections, construction errors, and loading effects. It is common knowledge that there is an interaction between the above two categories, in which the former may act as a catalyst to accelerate issues related to the latter and vice versa. As discussed in Section 2.3.2, the main loading effects on CSS structures are wind effects which cause vibration and fatigue problems. Due to the above issues, failures of a number of CSS structures have been reported worldwide, for example, Gallow et al. (2015) reported that an estimated 20 CSS structures fail every year in the USA.

Material-related deficiencies

Concrete is commonly used for the pedestals and foundations of the CSS structures. Common defects of the foundation include cracking, spalling, delamination, corrosion of steel reinforcement, and other material-related deteriorations including scaling, honeycombing, efflorescence and Alkali-Silica Reaction (ASR) attack.

Painted mild steel, galvanised steel, stainless steel, and aluminium are commonly used for the poles and arms of the CSS structure’s components. Common defects of these components include coating failure, corrosion, cracking, break or fracture, buckling and fire damage.

Fabrication and construction issues

Ghaedi et al. (2016) reported that the highest number of observed deficiencies on CSS structures were related to loss of galvanisation, corrosion, and missing hardware of the posts, frames and truss members. For the foundation, loose anchors, poorly constructed mortar pads and loose and missing hardware are also common. Most of the deficiencies reported include insufficient thread engagement, gaps in connection plates, and loose nuts. The lack of thread engagement may likely be due to construction issues, but it presents a major concern about structural integrity, as only a portion of the bolt was being used to secure the pole to the foundation. In addition, gaps in the connection plates may indicate the loss of pretension in the bolt or a bolt subjected to potential cracking.

Physical defects in the weld joints between the structural members due to fabrication issues have also been reported (Nims et al. 2019) and may lead to the initiation of cracks.

Fatigue defects

Fatigue is defined (AS 5100.6:2017) as ‘the process of initiation and propagation of cracks through a structural part due to action of fluctuating stress’, which is caused by cyclic loading. Due to fatigue, the material’s strength is degraded due to crack initiations and propagation until failure occurs. When the number of cycles of fluctuating stresses exceed the fatigue thresholds at critical structural details, even if the absolute value of the stresses is below the material’s ultimate strength, fatigue failure will eventually occur. In these circumstances, the structure has not been adequately designed for the fluctuating loads and was experiencing excessively large stress ranges. In practice, for CSS structures, failure/cracking due to fatigue is more likely to occur than due to overloading (Gallow et al. 2015).

As discussed in Section 2.3.2, most of the fatigue damage reported in literature is located at mast-to-arm connection, column base plate and anchor bolts, with the following common defects:

- cracked anchor rods both above and within the concrete foundation
- loose nuts and missing connectors from both anchor rods and structural bolts
- cracked and broken welds
- fractured tubes
- clogged drain holes, debris accumulation, and corrosion

- internal corrosion of tubular members (resulting in a reduced section)
- poor fit-up of flanged connections with cracking and missing bolts
- installation of signs exceeding design square footage, causing structure overload.

Some examples of failures of CSS structures are discussed below. The root causes of these failures typically include fatigue, overloading, and interaction of fabrication/construction defects with cyclic loading effects.

Morris and Thomas (2009) reported that an anchor rod within the base plate anchorage (Figure 2.6) for one gantry (carrying the Broderick Road sign), located near Geelong, Victoria, was completely fractured. The failure surface of the anchorage rod was found to have 'beach' marks which indicated that the mode of failure was fatigue. Outcomes from a monitoring program undertaken on this structure after being re-erected suggest that the failure was due to accumulation of fatigue damage as a result of significant vibration caused by wind-induced vortex shedding.

Figure 2.6: Base plate anchorage assembly

Source: Morris and Thomas (2009).

In NSW, a large cantilever sign structure collapsed onto the traffic carriageway near Blacktown in 2008 (Figure 2.7). An investigation was undertaken by the RTA (former TfNSW) into the failure consisting of ultrasonic testing and material tests of the failed anchor bolts. It was reported that the possible causes of failure included: (i) the lack of grout in recesses and under the base plate that resulted in a large unsupported length of the bolts, (ii) fatigue damage (indicated by ratchet marks and beach marks) in the bolts, and (iii) the as-installed bolts were significantly under strength, with nuts of a lower grade than as designed and the use of Grade 8.8 bolts which was not compliant (material test results indicated that the microstructure of the bolt was not compliant with AS 4291.1 for Grade 8.8 bolts which requires a quench and tempered (martensitic) microstructure).

Figure 2.7: Failure of a cantilever sign structure in NSW, 2008

a. Fallen sign structure

b. Failed anchors

Source: TfNSW.

Strong wind was believed to be the cause of a collapse of a cantilever sign structure in Whangaparaoa Road, Auckland in 2019 (Figure 2.8). The failure appeared to occur at of the base of the pole; however, no further details were reported.

Figure 2.8: Failure of a cantilever sign structure in Auckland, NZ, 2019

Source: https://www.localmatters.co.nz/news/34532-gantry-collapse-investigated.html, viewed 28 July 2020.

Nims et al. (2019) investigated a 4-chord truss CSS structure, located on Alum Creek Drive at the I-270 interchange near Obetz, Ohio, after the structure failed and was removed from service, to determine the causes of failure. On this structure, typical fillet welds were used for the connections between the chords and the diagonals. Figure 2.9 shows that the failure occurred at 2 different locations including the top chord and the weld between the chord and the diagonals. The top chord was completely severed, while the welds had cracked up, but the crack had not yet advanced through the chord. Both locations were near weld junctions of the diagonals and chord.

Analytical wind analysis carried out on this structure indicated that member stresses were within allowable limits, and the nominal fatigue life of the critical members exceeded the service life of the truss (typically 50 years). It was identified that the welds had physical defects due to fabrication issues that led to the initiation of cracks. Under cyclic loading, crack propagation occurred within the high stress regions.

The paper reported that there was an interaction between an instantaneous material failure due to overloading and a slower material failure due to fatigue. The former occurred on a small smooth surface area and resulted in the initiation of cracks, while the latter occurred on a much larger uneven and irregular surface. The most severe fractures were located along or adjacent to the welds connecting the cross members to the chords.

Figure 2.9: Failure of cantilever truss members

a. Fracture of top chord

b. Close-up of fracture of chord

c. Weld cracking

Source: Nims et al. (2019).

Furthermore, Figure 2.10 shows the failure of a cantilever structure in Florida after a hurricane (Cook & Halcovage 2007). The circular concrete foundation failed as a result of an applied torsion which caused a concrete breakout failure due to shear within a plane parallel to the edge of the anchors. It was obvious that this failure was due to overloading.

Figure 2.10: Failure of a cantilever sign structure in Florida due to hurricane

a. Fallen post

b. Close-up of foundation concrete breakout

Source: Cook and Halcovage (2007).

Florea et al. (2007) reported a falling cantilever highway sign structure in Pflugerville, Texas due to fatigue failure resulting from overstressing, poor welding quality, and low fatigue strength (Figure 2.11).

Figure 2.11: Failure of a cantilever sign structure in Pflugerville, Texas

a. Fallen cantilevered arm

b. Fatigue failure at mast-arm connection

Source: Florea et al. (2007).

On traffic signal structures in Wyoming, cracks were primarily located along the sides of the built-up box, at the mast-arm-to-post/pole connection where the fatigue resistance is relatively poor (Dexter & Ricker 2002). The cracks had propagated through the pole wall at the toe of the weld connecting the built-up box. When galloping and natural wind loads were applied to the structure in accordance with The National Cooperative Highway Research Program (NCHRP) Report 412 (Kaczinski et al. 1998), the box-to-column connection was overstressed by factors of 3.62 and 2.43 in the in-plane and out-of-plane directions, respectively. In addition, the column section below the box connection was overstressed by a factor of 2.68.

The following are worth noting regarding fatigue of CSS structures:

- Testing has indicated that the primary effect of constant amplitude loading is within the live-load stress range (Consolazio et al. 1998), and the constant dead load effects do not affect fatigue (Garlich & Thorkildsen 2005).

- For a particular structural detail, the strength and type of steel have a negligible effect on its fatigue resistance (Consolazio et al. 1998). In addition, the welding process and minor deviations in weld quality, while important, also do not typically have a significant effect on the fatigue resistance (Garlich & Thorkildsen 2005).
- Large-amplitude vibration may not cause a structural problem; however, when the displacement range exceeds 200 mm, motorists cannot clearly see the signals or signs and are concerned about the safety of the vibrating structures (Kaczinski et al. 1998).
- Fatigue cracking of various details (e.g., anchor bolts, column base details, and mast-arm-to-column connection details) would eventually occur due to the high stress ranges associated with the vibration (Kaczinski et al. 1998).
- There are many cases of severe fatigue cracks that are discovered before the cracking leads to collapse. The economic cost of inspecting, repairing, or replacing nuisance cracks is substantial (Dexter & Ricker 2002).

2.3.4 Inspection Regime

Types of inspection and frequency

There exist many variations in the types of inspection implemented for CSS structures (e.g., Ghaedi et al. 2016, Bicilli 2015, and Garlich & Thorkildsen 2005, VDoT 2014); however, the most common inspection regime includes initial/inventory inspection, regular inspections and special inspections:

- Initial/inventory inspection is a visual inspection, typically carried out for new or replacement structures to provide an inventory database and establish a baseline for the as-built condition of the structure.
- Regular inspection is a visual inspection, carried out at a specified interval, to detect any sign of deterioration and changes in the condition of the structure in comparison to the previous inspections.
- Special inspection is a hands-on or in-depth inspection, carried out to further investigate structural deficiencies triggered from the regular inspections, to identify damage to a structure due to impact or collision or following a natural disaster event. Special inspection is also used for construction acceptance or following the installation, repair or replacement activities.

Visual, ground-based inspection is the most common type of inspections for initial and regular inspections due to difficult access conditions which usually require traffic control. Non-destructive testing (NDT) may be performed as a complementary tool to the inspection process, as part of the standard inspection process, or if visual inspection identifies a need to perform them (Ghaedi et al. 2016).

Hands-on or in-depth inspection is defined as an inspection procedure that includes arm's-length inspection of all major components. The hands-on procedure may often include the need for traffic control to access the structure as well as the use of one or more NDT methods to assess the structural integrity of one or more components, such as fatigue prone details.

In addition to the above inspections, some jurisdictions such as Virginia Department of Transportation, USA (VDoT 2014) also undertake regular base inspections which focus on the structure's base and anchor bolts, to detect any deterioration of these components.

Non-destructive testing techniques

Non-destructive testing techniques (NDT) are important tools used for the inspection of CSS structures to identify the deficiencies that cannot be detected by visual inspections. Examples of the defects found through NDTs include small fatigue cracks in welds, cracked anchor rods, and corrosion of the interior of the structural element (Garlich & Thorkildsen 2005). The following are the most common NDT methods used for CSS structures:

- Ultrasonic thickness 'D meter' measurement is the most critical NDT for CSS structures to detect the corrosion of the interior of structural members that cannot be visually observed. A simple ultrasonic thickness gauge can also be used to detect possible fractures and check the lengths of anchor rods. Possible cracks or other flaws in the anchor rods can be detected by more sophisticated ultrasonic flaw detection equipment.
- Dye Penetrant Test is the most common NDT method for examining cracks in exposed welds of steel structures and non-ferrous materials such as aluminium. While Dye Penetrant can only detect surface cracks, this test can be used to confirm visual observation of a crack and also to distinguish between cracks and surface defects such as galvanising flaws.
- Magnetic Particle Test is commonly used for examining cracks in exposed welds of metallic materials such as steel. This test can detect surface cracks and near surface cracks.
- Eddy Current Test is used for painted steel members or aluminium structures to detect weld cracks where magnetic particle testing does not function due to lack of magnetic attraction.

Inspection frequencies

For CSS structures, inspection frequencies are specified depending on several factors including material type, structural redundancy, and traffic control requirements. It is common that regular inspections are carried out every 4–6 years (e.g., VDoT, Kansas DoT, Ohio DoT, and France). Additional inspections are carried out with a shorter interval, for example, VDoT carried out base inspection every 2 years (VDoT 2014), appraisal inspections are carried out in France every 3 years (Bicilli 2015).

For example, in France, inspection types and frequencies are similar to bridges which include (Bicilli 2015):

- annual routine inspection: a visual, ground-based inspection to identify any potential safety problems
- appraisal inspection: a visual, ground-based inspection carried out every 3 years to identify if there are any members or details that need repairing
- detailed inspection: a periodic in-depth inspection, scheduled every 6 years as a maximum and carried out by specialists with full access to the structure's components.

Priorities are usually given to the inspection of non-redundant cantilever sign structures in France, CSS structures of greater than 20 years old, or structures where sign panel sizes exceed those originally designed for. It is common that aluminium sign structures have a regular inspection frequency not to exceed 2 years.

Rating criteria/system

In general, 4 condition states are used to rate the CSS structure's components. For example, the rating system presented in Table 2.3 is recommended by Garlich and Thorkildsen (2005), which is similar to the condition rating system used by most road agencies in Australia for road structures.

Table 2.3: Element condition rating

Rating	Description	Action
0	Not applicable	None
1	Element performs intended function with high degree of reliability (Good)	None
2	Element performs intended function with small reduction in reliability (Fair)	Repair element, increase inspection frequency, do nothing
3	Element performs intended function with significant reduction in reliability (Poor)	Repair or replacement of element within specified time frame

Rating	Description	Action
4	Element does not perform intended function with any degree of reliability (Critical)	Immediate repair or replacement of element

Source: Garlich and Thorkildsen (2005).

Similar to bridge structures, the CSS structures are also divided into a number of component groups. The following component group classification is used commonly among the US Departments of Transportation (Rowe 2005):

- foundations
- anchor bolts
- base plates
- column supports
- column to arm/chord connections
- arm/chord members
- chord splice connections
- span truss members
- sign frames
- sign panels
- catwalks
- luminaires
- sign attachments
- slip joints.

Detailed descriptions of each component group and the associated criteria for condition state ratings are usually provided in various jurisdictions' inspection guidelines (e.g., VDoT 2014, Garlich & Thorkildsen 2005).

2.3.5 Testing and Monitoring

Outcomes of wind-tunnel tests to evaluate galloping and vortex shedding on scale-models of CSS structures were reported by Kaczinski et al. (1998). The key conclusions made regarding the dynamic response of CSS structures to the galloping and vortex shedding phenomena include:

- Galloping effect is very sensitive to the site-specific conditions and does not occur frequently. Factors that affect galloping include the dynamic properties of the structure, aerodynamic properties of the attachment details, and characteristics of the flow. Two identical structures located closely to each other may experience significantly different galloping magnitude and frequency. In some cases, the risk of galloping and excessive deflection and fatigue failure may be acceptable; however, it was recommended that structures located above high-volume high-speed roads should be designed to resist galloping-induced fatigue damage.
- CSS structures are most susceptible to galloping-induced oscillations when the signal attachments are rigidly mounted on the mast arm and are configured with backplates.
- Once the galloping instability is initiated, lower velocity winds can still cause resonant vibrations which result in damaging stress cycles. In addition, the resulting resonant vibrations increase with increases in the flow velocity. Observations have been made for structures in the field that they continue galloping despite gusts and the range of vibration increases as the velocity increases.
- Limited test results show that CSS truss structures are less susceptible to galloping-induced vibration.
- CSS structures are most susceptible to vortex-induced vibrations due to the shedding of vortices from the horizontal members. Therefore, vortex shedding from the column does not appear to be significant

enough to excite structural vibrations. In addition, only structures with horizontal arms of relatively large diameter are susceptible to vortex-induced vibration.

- When the critical wind velocity falls significantly below 5 m/s, the design of CSS structures for fatigue may not need to consider vortex shedding.
- The addition of attachments to the horizontal members of CSS structures appears to disrupt the correlation of the vortex shedding forces along the span. As a result, vortex induced vibrations need only be considered prior to the installation of attachments to the structure.

Hosch et al. (2017) undertook a controlled test program of a CSS structure to investigate the effects of truck-induced wind gusts. Normal- and shear-strain deformations around the circumference of the post in proximity to the base-plate weld were measured to capture the in-plane and out-of-plane flexural motion and torsion of the post. Bending and torsion deformation were measured using uniaxial strain gauges and 45° rosettes, respectively. Test results were compared with AASHTO's truck-induced wind-gust design fatigue load (AASHTO 2015). The following observations were reported:

- The normal strain from the AASHTO truck provision was conservative and did not reflect the deformation response behaviour of the cantilever structure.
- The experimental flexural behaviour from truck gusts indicated a combined effect of horizontal and vertical deformation.
- A significant torsional strain deformation was captured, which indicates the existence of a significant horizontal truck-gust component.
- Torsion of the post created shear and bending stresses on standoff anchor bolts, which resulted in large anchor-bolt stresses due to truck gusts.

Morris and Thomas (2009) also reported the outcomes of a short-term in-service monitoring program undertaken in 2007 on the Broderick Road cantilever sign gantry (Victoria, Australia). The structure has a 12.3 m cantilever out-stand, designed to the 1992 Austroads design code, and constructed in 2000. Structural defects in gantry base plate anchor rods were discovered in 2007 due to fatigue. Two sets of instrumentation were installed, including (i) vertical and lateral direction accelerometers located at the cantilever tip; and (ii) vertical, lateral and longitudinal accelerometers located 2.3 m from the column.

It was observed from the tests that vortex shedding was significant on this particular structure. A link was also observed between vortex shedding from the vertical column and lateral vibration of the cantilever tip when the wind direction was parallel to the sign face.

According to Morris and Thomas (2009), fatigue failures in the anchor rods were not easily detected by NDT testing; therefore, monitoring of cantilever sign gantries for fatigue damage is not an effective means of managing the risk of failure. Since the residual risk of failures in the base plate anchorage for this type of structure designed prior to 2007 is high, a different approach to managing the risk should be developed.

It was recommended that vortex shedding be considered in the detailing of cantilever sign gantries. As the phenomenon appears to be very specific to wind direction, attaching strakes to the column face on the sides parallel with the face of the sign may help to mitigate the problem by removing the source of lateral excitation of the cantilever tip.

2.3.6 Mitigation Measures

Fatigue stresses and vibrations of CSS structures can be mitigated by a number of measures including control of the fatigue resistance (e.g., increasing the stiffness of the structure), changing aerodynamic characteristics of the structure, and using mechanical dampers to reduce the intensity of oscillation (Garlich & Thorkildsen 2005). Furthermore, as discussed in Section 2.3.2, different vibration mitigation techniques can be used for different wind loading phenomena to reduce fatigue stresses, including (Fouad et al. 2003):

- for galloping, changing the dynamic characteristic of the structure and aerodynamic properties of its attachments

- for vortex shedding, changing the dynamic properties and aerodynamic characteristic of the structure
- for natural wind gusts, increasing damping (by using dampers) and stiffness (by increasing member sizes and changing geometry) of the structure
- for truck-induced wind gusts, increasing the vertical clearance of the sign panel and reducing the effective horizontal projected area (e.g., by using perforated panels and open grating).

Control of the fatigue resistance

Control of the fatigue resistance involves control of the nominal stress range and the fatigue threshold of the details. CSS structures should be designed so that the stress ranges due to the fatigue design loads are less than the fatigue thresholds for each structural member or detail so that fatigue will not occur even for a large number of applied load cycles (Garlich & Thorkildsen 2005). The current AASHTO specification (AASHTO 2015) specifies an infinite number of applied load cycles to be used in analysis. The stress ranges can be controlled by using high-strength materials, changing the stiffness of the structure (Dexter & Ricker 2002), or using a damping system (Epp et al. 2019).

Use of high-strength steel (higher than 420 MPa yield strength) typically is not cost-effective since the fatigue resistance of various grades of steel and anchor rods is the same (Garlich & Thorkildsen 2005). In fact, using high-strength steel can decrease the dimensions of the members or anchor rods for the same design loads, but the stress ranges under the fatigue design loads in the members will increase, which make fatigue more problematic.

The stiffness of the structure can be altered by changing the members' sizes or changing the structure's layout. Increasing stiffness of the structures is reported to be effective in the control of fatigue in CSS structures (Gallow et al. 2015) because the natural frequency of the structure is increased, thus reducing fatigue stress due to wind load effects. For 4-chord trusses, altering post size or stiffness could improve the structure's natural frequency and fatigue stresses generated at its base. However, Gallows et al. (2015) notes that alterations in truss type, depth, and width for the horizontal support members were not significant in changing the fatigue resistance at the base of the post.

It is noted that the shape of the member may also have favourable effect on fatigue caused by wind loads, for example, use of circular members can mitigate fatigue stresses caused by natural wind gusts and truck-induced wind gusts (Gallow et al. 2015).

Changing aerodynamic characteristics

The aerodynamic characteristics of CSS structures can be changed by adding damping plates and louvered backplates to the main members of the structure. An example as recommended by Morris and Thomas (2009) is to attach strakes to the sides of the pole, in a way similar to the treatment for tall chimneys, to mitigate the effects of vortex shedding. A mixed track record, however, was reported for the effectiveness of this method (Dexter & Ricker 2002).

Use of mitigation devices

It is expected that effective mitigation devices installed on a CSS structure can result in a reduced vibration amplitude, which leads to a smaller stress range induced in the structure. Consequently, the structure is less susceptible to potential fatigue failure, in addition to a reduction in fatigue requirement for the structure (Epp et al. 2019). The use of a number of mitigation devices has been reported, such as wind deflectors, Stockbridge-style dampers and a Valmont Mitigator; however, use of these devices is not widely spread. Dampening devices are viewed as contingency items that may improve the performance of the structure but are not so reliable that their inclusion can alter the design. There have been concerns on the service life and efficacy of the mitigation devices (Epp et al. 2019). Insufficient evidence has been provided in literature on whether the devices are able to work effectively over the service life of the structure and if they are effective over a diverse range of environmental conditions. Further research is needed to address these concerns.

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.

2.4 Summary

In summary, the following sections present the key learnings regarding the current practice in managing CSS structures in Australia and internationally.

2.4.1 Australian Practice

The following comments are made regarding Australian practice:

- Only a few failures of CSS structures have been reported in Australia. The low failure rate may be due to the following:
  - long-lever CSS structures are not used in most road agencies
  - good inspection and maintenance practices are currently in place so that defects are detected early and are timely rectified.

Conditions of CSS structures are maintained so that sudden failures are not likely to occur. Of the failures reported, the main cause of failure was fatigue. With regards to current guidelines and risk management measure for CSS structures:

- Australian road agencies have technical guidelines and specifications for design, construction and inspection of CSS structures. The inspection regime of CSS structures is typically similar to that of other road structures. Australian road agencies currently have a more stringent inspection regime than its US counterparts, with more frequent regular visual inspections (Level 1 and Level 2).
- Risk management of CSS structures has been the focus of some road agencies such as DoT Victoria through a regular inspection regime and measures to mitigate potential risks of collapse and additional risks to third party road users.

2.4.2 International Practice

A number of failures of CSS structures have been reported overseas. The main causes of failure include material deteriorations, overloading, and fatigue. For CSS structures, failure and defects due to fatigue are more likely to occur than failure due to overloading. Fatigue defects in CSS structures usually occur at the connection of the mast arm to the pole, at the base of the pole, at the top of stiffeners, at the perimeter of hand holes and at anchor rods.

Fatigue in the CSS structures is caused by wind-induced vibrations due to 4 wind loading phenomena including natural wind gusts, galloping, truck-induced wind gusts and vortex shedding. Each wind phenomenon may cause different types of structural responses on the structures depending on structural configurations, location, and site-specific conditions.

CSS structures are susceptible to wind-induced vibrations due to having a low natural frequency and low damping as a result of lightweight design and high structural flexibility. As the structure’s natural frequency is within the common frequency range of wind load phenomena, resonance is likely to occur, which causes large vibration and stress range in the structure.

It was found that most road agencies implement an inspection regime that is similar to that applied to road structures; regular visual inspection is most common and is supplemented by in-depth, detailed inspections and NDT methods. Given that fatigue defects occur frequently to the base components of the CSS structures, additional inspections are also carried out by some road agencies which focus on these components.

Due to the fact that fatigue defects due to wind-induced vibrations develop over a long period of time, short-term monitoring of CSS structures for fatigue defects is not an effective means of managing the risks of failure. A long-term monitoring program may be able to determine the wind patterns for a specific site and if large vibrations or large stress ranges occur in the structure.

Mitigation measures have been implemented, including control of the fatigue resistance of the structure, changing aerodynamic characteristics of the structure, and using mechanical dampers to reduce the intensity of oscillation. However, while these measures are typically applied to improve new designs, limited literature on retrofitting existing CSS structures has been reported.

There has been limited information about the issues related to the use of imported materials.

3 Review of Current Main Roads Practice

3.1 Inventory Database

The information presented in this section is based on an Excel file database provided by Main Roads on 11 August 2020.

Main Roads currently manages 56 CSS structures on its network, in addition to 8 long-lever (between 16 and 22 m) cantilever sign structures which have been constructed as part of the Smart Freeways – Kwinana Northbound Project. These 8 long-lever structures are yet to be entered into the database.

Excel spreadsheets are currently used for the database, which generally provides basic inventory information, including structure ID, name, location (longitude and latitude), construction date, length, clearance, last inspection date and type, and next scheduled inspection date. The data regarding the individual structures, however, was not easy to access or unavailable, and the database did not provide an overview of the changes in a structure's condition over time.

3.1.1 Design Information

Currently, no information about the design standard of each CSS structure is included in the inventory database. This information is important for the load rating assessment of the structures during its service life when a need arises.

Cantilever Signs: Sign Structural Design Guide (Main Roads 2015) specifies that 'The structural analysis is to be carried out in accordance with Australian Standards and Code'. It does not specify a specific version of any standards; therefore, it is likely that the construction date of a structure could be used as a basis to determine the design standard. For example, structures built in 2000–2005 were likely designed using the 1992 Austroads *Bridge Design Code*; and post-2005 structures using AS 5100 (2004).

It is worth noting that the concept design for type B and C VMS gantry (Main Roads 2016) has adopted additional load combinations for fatigue (natural wind gust, galloping, truck-induced gust) from the AASHTO *Standard Specifications for Structural Supports for Highway Signs, Luminaries and Traffic Signals* (2009) and Interim Revision (2011).

Because fatigue defects are critical to CSS structures, AS 5100.2:2017 refers to the current AASHTO standard (AASHTO 2015), which requires the use of a more conservative fatigue design criteria – constant amplitude fatigue threshold (CAFT). The CAFT is defined as 'nominal stress range below which a particular fatigue detail can withstand an infinite number of repetitions without fatigue failure', meaning an infinite fatigue life. The DoT also provide several design supplementary provisions (VicRoads 2018). These updates should be taken on board in the future revision of *Cantilever Signs: Sign Structural Design Guidelines* (Main Roads 2015).

3.1.2 Material Data

No information is recorded in the database regarding the materials used, together with material testing records, which may be available on archived drawings.

It is understood that material test results are available for the structures on the Smart Freeways – Kwinana Northbound Project. Due to the limited availability of large diameter steel sections required, steel sections were fabricated from a variety of steel grades certified to different international standards. Independent testing was undertaken in Australia to corroborate the material properties shown on mill certificates. However, details of the issues and concerns on the imported steel materials are not available for review.

3.1.3 Inspection Reports

Previous inspection reports lack details regarding the components and observed deficiencies. Therefore, a comparison cannot be made between subsequent inspection reports to determine if the deficiencies observed in the field previously existed or are new. Such a comparison is necessary to observe any trends related to degradation over time or the impact on structural integrity of the structures.

In particular, the following data/information was missing or deemed incomplete:

- overall span length data (34 structures)
- historical records such as as-built plans, material testing, maintenance, repairs, modifications, and installation of new sign panels
- design standard (which can be estimated based on construction date)
- overall condition rating
- last Level 1 inspection date
- last Level 2 inspection date (and also previous inspection dates)
- last Level 3 inspection date (no information is currently included in the database)
- material
- deficiencies.

3.1.4 Inspection Data

Except the recently built structures, the age profile of Main Roads' CSS structures ranges from 2000 to 2017, with a large number of structures constructed in 2009 and 2016.

A condition state profile is not readily available in the current database, as there is no information on the overall condition state included. The 19 sample Level 2 inspection reports show that most components are in CS1 or CS2, a very small number of non-principal structural components are in CS3, and no components are in CS4.

A Q&A email exchange between ARRB and the Main Roads project team regarding the inspection database of CSS structures is summarised as follows:

> Q: The current database shows that some Level 1 inspections were last conducted in 2018 – are they not conducted annually?
>
> A: Level 1 inspections are carried out annually for each CSS structure; however, inspection reports are not always entered into the database system by the regional inspectors in a timely manner.
>
> Q: Are previous inspection records available? Since most structures were built in 2009 and 2016, are handover inspection reports available? For those structures built in 2008–09, with a 7-year cycle, why the last inspection was 2018–19 while they should have been conducted in 2015–16?
>
> A: There are packages of gantry inspections in progress at the moment; however, there have been delays associated with COVID-19. These are the first of the Level 2 inspections. Note that our policy has the first inspection at 10 years from completion of construction.
>
> These handover inspection reports are provided to the regional asset managers. Handover reports of structures on Smart Freeways are not available yet.
>
> Q: Are there any records of any Level 3 inspections undertaken in the past?
>
> A: The current gantry inspection packages are a combined Level 2/Level 3.
>
> Q: Is asset value data available? It will be needed for the benefit-cost analysis (BCA) later, and also to strengthen a business case if any enhanced management program is recommended.
>
> A: The asset valuation data is available for sign support structures.

In general, inspection data are available but have not been entered into the database.

3.2 Current Inspection Practice

The information presented in this section is based on Main Roads' *Sign Gantry Guidelines (Level 1 and Level 2 Inspections)* (Main Roads 2013a).

3.2.1 Types of Inspection and Frequency

Level 1 inspection (Level 1)

Level 1 inspections are carried out annually, or after an event (e.g., large storm, impact damage) occurs that may impact the condition of the structure. The purpose of an Level 1 inspection is to check on the overall safety and performance of the structure and to identify any major accident damage or incident and any obvious failure of structural components. It also checks that maintenance works are being carried out.

Level 2 inspection (Level 2)

Level 2 inspections are carried out every 7 years, which include visual inspection and specialist testing of major components (weld inspection, measuring of steel thickness and protective coatings using relevant NDT methods). An Level 2 inspection is a detailed inspection where all components are inspected within 1.0 m with the aid of an elevated work platform and lane closure(s) as required. In some instances, access ladders/walkways have been used, which do not provide access to the bottom chords, etc. All components of the gantry structure above ground and footings, anchor bolts and mortar grout pads within 500 mm of ground level are included in the scope of Level 2 inspections.

It is noted that a checklist for what to look for on each component is provided in the structure's inspection guidelines (Main Roads 2013a).

Level 3 inspection (Level 3)

Special inspection and investigation (L3) are carried out when necessary, e.g., being triggered by findings from L2 inspections, and for gantry columns and foundations located more than 500 mm below ground level.

As indicated in Section 3.1.4, no separate L3 inspections have been conducted on Main Roads' CSS structures.

3.2.2 Use of NDT in Inspections

It is specified in Main Roads (2013a) that NDT be used for all L2 inspections. The following common NDT tests have been used as reported in the L2 inspection reports:

- Eddy current test, to inspect welds (all welds are inspected to detect the presence of cracking). It should be noted that of the 19 L2 inspection reports reviewed, this test was only used to inspect the welds of structure ID 8022, while the welds on the remaining structures were examined by visual inspection.
- Ultrasonic test (UT), to measure thickness of components. Some interpretation of the thickness measurement results is provided in the inspection reports, but does not show comparisons with design and/or previous inspection report.
- Dry film thickness (DFT) test, to assess thickness of protective coatings.

3.2.3 Structural Inventory Classification

Main Roads' structural inventory classification for CSS structures (Appendix A of Main Roads 2013a) is, in general, aligned with international practice, i.e., grouping of components: barriers, beams, columns, footings, anchor bolts, mortar grout pads, maintenance access, and signs. A number of components are classified within each group (by name of component), instead of a component identification numbering system (such as used by TMR). For example, beams and connections are included under 'Gantry Beams'. Bracing components appear to be missing from the component groups listed; however, they were included in recent Level 2 inspection reports. Given there are a limited number of components, it is justified not to use a component ID system.

Fatigue of column anchor bolts due to cyclic wind loading has historically been identified (Main Roads 2013a) as one of the major failure mechanisms of sign gantries. As such, the anchor bolts and mortar grout pads are important components to be checked during an inspection.

3.2.4 Condition State Classification

Four condition states are provided in the manual (Main Roads 2013a) with detailed descriptions for each component based on materials (steel, concrete, timber), and component types, including:

- CS1: good condition with little or no deterioration
- CS2: generally in good condition, with some minor deterioration
- CS3: poor condition, with moderate deterioration
- CS4: very poor condition, with severe deterioration.

While the criteria for rating for concrete components are specific and easy to follow, they are generally qualitative and subjective for steel components. Some other jurisdictions use a more specific criteria for condition rating, especially for the anchor bolts, nuts and washers, such as the one presented in Table 3.1.

Table 3.1: Condition state classification

Rating	Condition	Description
1	Good	Some minor issues or nearly new; no repairs needed
2	Fair	Minor deficiencies (e.g., loss of galvanisation, corrosion, cracking, spalling); minor repairs needed
3	Poor	Major deficiencies (e.g., large-scale corrosion and section loss, fatigue cracks); repair or rehabilitation needed
4	Critical	Potential impact to structural integrity (more than 50% loose or missing anchor nuts, more than one cracked anchor bolts); major rehabilitation or replacement needed

Source: Gostautas et al. (2015).

Another example of specific condition rating criteria for a critical condition (equivalent to CS4) as used by VDoT (2014) is presented as follows:

Anchor bolts

The following classifications were used for the condition of anchor bolts:

- 30% or greater section loss of one or more anchor bolts.
- Any anchor bolts that are broken, sheared, or cracked.
- Any anchor bolt(s) having any relevant indications detected by ultrasonic testing.
- 2 of 4 top or levelling nuts loose or missing (luminaire not on a transformer base).
- Excessive movement of the transformer base or 2 of 4 anchor nuts seen moving when the pole is rocked.
- 1 of 4, 2 of 6, or 3 of 8 top or levelling nuts loose or missing (high mast light, camera, cell tower).

- 3 of 4 top or levelling nuts inadequately tightened (luminaire not on a transformer base).
- 2 of 4, 2 of 6, or 3 of 8+ top or levelling nuts which are not fully seated with a gap under the nut which has a height of 4% of the anchor bolt diameter or greater.
- 2 of 4, 2 of 6, or 3 of 8+ out of plumb anchor bolts which have a slope equal to or greater than 1:40.
- 2 of 4, 2 of 6, or 3 of 8+ anchor nuts that are less than 75% engaged.
- 1 of 4, 2 of 6, or 3 of 8+ top or levelling flat washers are missing where slotted/oversized holes exist and the nuts are embedded into the hole.
- 2 of 4 transformer base to anchor bolt connections are not fully engaged, are misaligned, or are undersized.
- Any breakaway couplers broken, sheared, or cracked.
- Transformer base cracked or broken and in danger of collapse.

Poles and base plates

The following classifications were used for the condition of poles and base plates:

- Structural members having one or more areas of 25% or greater section loss (20% or greater for weathering steel).
- Any cracks in the base plate, vertical stiffeners, hand hole, pole, welded joints, slip joints, or base plate or vertical stiffener to pole weld.
- Impact damage to any structural member in which the member is in danger of falling.
- Loose, missing, broken, or heavily deteriorated attachments to the pole (signs, cameras, sensors, etc.) in which the attachments are in danger of falling.
- 3 of 4 transformer base to pole nuts are less than 75% engaged.
- 3 of 4 transformer base to pole bolts are loose or missing.

Superstructure

The following classifications were used for the condition of superstructure components:

- Structural members having one or more areas of 25% or greater section loss.
- 25% or more bolts or nuts in chord to pole connection are loose or missing.
- Any cracks in the arm to pole connection plates.
- Impact damage to any structural member in which the member is in danger of falling.

3.3 Level 2 Inspection Reports

Seventeen Level 2 (Level 2) inspection reports of Main Roads' CSS structures were provided for review. Key outcomes of the review are presented below.

3.3.1 General Observations

The following general observations were made:

- The inspections have been carried out by qualified personnel.
- Full access for inspection was not provided on a number of structures, in which access was provided only by existing access ladders and walkways. Therefore, a close-range visual inspection was not possible for a number of components, such as the bottom chords of the gantry. However, no evidence was found that indicated the need for full access for all structures.

- NDT were used on all Level 2 inspections, with the most popular NDT techniques are UT test and DFT test. Eddie current test was used on only one structure (refer to Section 3.2.2). Visual inspection of the welds was carried out to identify the need for NDT inspections.

3.3.2 Common Defects

No critical deficiencies were reported. The most common observed deficiencies were related to corrosion, loss of galvanisation and welding imperfections as summarised below:

- Minor surface corrosion especially at the inside face of the panel where the panel abuts against the walkway longitudinal member, due to trapped moisture. Loose tek screws on walkway handrail panels.
- Minor corrosion of gantry beam or weld (only on 8066, 8112).
- Open holes on steel components of signs and walkway.
- Open hole (galvanising hole) on the bottom cord of 8061, 8062.
- Some minor cracking on concrete footings.
- 8022: at the bottom beam to column connection, half the bolts appear to be of a smaller size due to clash with welded connection; surface corrosion on bolt heads.
- Uneven galvanising thickness (8066).

It should be noted that no gaps or missing lock washers in anchor bolts have been reported.

3.3.3 NDT Test Outcomes

Common outcomes of NDT tests on the reviewed L2 inspections include:

- No cracking has been detected using Eddy current testing on any of the welds of the structures inspected. Minor imperfections (e.g., weld splatter) were observed via visual inspections. For example, structure ID 8066 has the most defects in welds, mainly due to construction quality.
- Thickness measurements suggest no section loss on steel section. For example, structure ID 8022, UT test found no reduction in thickness on all steel members.
- It was reported that the minimum galvanising thickness measured by DFT testing can provide an extra long-term life to first maintenance (25+ years).

3.3.4 Recommended Maintenance Actions

Maintenance activities recommended in the current L2 inspection reports are minimal, which mainly include treatment of protective coating, surface corrosion and monitoring. This indicates that Main Roads CSS structure stock is in good condition, which is well-aligned with their young ages. As an example, the following maintenance actions are recommended for Structure 8022, which was built in the year 2000 (one of 3 oldest structures on the network):

- Control corrosion/replace anchor bolts of bottom beam to column connection.
- Re-coat beams.
- Monitor surfaces with surface corrosion and minor pitting of all sign mounting poles at future inspections.
- Monitor surface corrosion and minor defects to sign mounting bolts at future inspections.

3.4 Maintenance, Rehabilitation and Renewal

There was no data/information available for review, possibly because Main Roads CSS structures are of young ages and the fact that the maintenance actions have just been recommended in the recent Level 2 inspection reports. The following comments are made:

- Apart from the handover inspection (report managed by regional asset manager), only one Level 2 inspection has been undertaken due to a 7-year cycle, and Main Roads policy of carrying out the first Level 2 inspection at 10 years after construction.
- Intervention levels and required actions are determined based on the outcomes of Level 2 inspections, including maintenance, repair and Level 3 inspections, particularly for components having condition state 3 or 4.
- No Level 3 inspections have been triggered/undertaken separately. Currently, Main Roads conduct combined Level 2/Level 3 inspections.
- No major rehabilitation works have been carried out (e.g., replacement of components, connections, paint work).
- Maintenance works included tightening of loose bolts and fasteners of signs and patch painting.
- There is insufficient data to assess re-coating frequency.

3.5 Risk Management

No specific information about risk management practice in Main Roads for CSS structure was available for review; however, it is understood that currently Main Roads manages risk related to CSS structures through the Level 1 and Level 2 inspection regime. Potential risks are identified through these inspections. Risk mitigation measures will be implemented through programmed maintenance activities.

3.6 Structural Monitoring Proposal

A number of questions need to be clarified at the commencement of the development of a monitoring program:

- What are the aims of the monitoring program?
- Why do we need to monitor the structure?
- What are the critical locations and what needs to be measured?
- How do we use the outcomes of the monitoring program in the asset management decision-making process?

The following considerations should be taken into account in the development of a monitoring program for CSS structures:

Aims

The aims of a structural monitoring program include:

- assessing the long-term performance of the structure through time and identifying any factors that affect the structure's performance.
- providing risk assurance for the CSS structure by monitoring the changes in the performance and behaviour of various components of the structure through time
- establishing a knowledge base on the fatigue behaviour of these structures under the effects of various types of wind loads

- providing specific information and basis for decision making on the implementation of appropriate risk mitigation measures
- implementing the learnings from this program on other similar structures on the network.

Objectives

The objectives of a monitoring program comprise:

- developing a cost-effective instrumentation plan for long-term monitoring of Main Roads' CSS structures that is fit-for-purpose, i.e., aligns with the monitoring objectives
- selecting a suitable data acquisition system including data logger and sensors
- providing guidelines for installation and calibration of the system
- providing a methodology for data collection and processing
- interpreting the acquired data to form the knowledge base of the behaviour and performance of the structure.

Instrumentation plan

The following considerations should be taken into account when preparing an instrumentation plan:

- Data logger: should be robust and can work reliably for a long time. A wireless data logger is essential for a remote site, which can communicate via a mobile network. The monitoring data can be continuously accessed via a cloud service. The data logger should be installed in a secured site box to prevent vandalism.
- Power source: solar power is the best option for a remote site where grid electricity is not available.
- Strain measurements: the sensors should be of a suitable type for long-term usage, i.e., they are less influenced by environmental factors such as electromagnetic field, moisture and temperature. Fibre optic sensors would be a good option. The strain sensors should be installed at critical sections of principal structural components where high stress variations and magnitude are expected.
- Dynamic response: accelerometers can be used to measure the dynamic response of the structure. Depending on the vibration modes, uni-axial or tri-axial accelerometers can be installed where high deflections are expected.
- Wind speed and direction: anemometers can be used to measure wind speed and direction. They can be installed on top of the cantilevered arm.
- Effects of passing-by trucks: cameras can be used to capture images of big trucks passing under the CSS structure. This information can be used to relate the truck-induced wind effects with truck types.

Monitoring scheme proposed for the Smart Freeways Project

A schematic instrumentation plan proposed for the monitoring of CSS structures on the Smart Freeways Project was provided for review. The proposed monitoring system includes data logger and sensors installed on a cantilevered single chord arm structure, comprising:

- a data logger installed on the cantilever arm near the pole
- uni-axial and rosette strain gauges installed at various points at the base of the pole and at the pole-arm connection on both the pole and the arm to monitor strains at these locations
- tri-axial accelerometers installed at midspan and tip of the cantilever arm to measure vibration
- a camera installed at midspan of the arm to monitor traffic lanes
- an anemometer installed on top of the arm at midspan to measure wind speeds.

The sketch included in Appendix A provides a concept design of an instrumentation system for a CSS structure, i.e., it specifies the number, type, and approximate locations of sensors that are required for a monitoring scheme. This instrumentation plan can capture the most critical strains and vibration at critical locations and components of the structure. In addition, the anemometer and camera can provide wind and traffic data. The long-term performance of the system, however, needs to be considered to ensure that the system can work reliably for at least 12 months. Depending on the structure's type of a selected CSS structure, the number and locations of each sensor type will need to be revised.

3.7 Benchmarking Main Roads Practice with Australian and International Practices

3.7.1 Design Specifications

Main Roads (2015) should be updated with the changes in AASHTO (2015) and AS 5100.2:2017, taking into consideration the design improvements in VicRoads (2018) to ensure that current advancements in the design of CSS structures are considered, especially the provisions for fatigue design. Refer to Section 3.1.1 for more details.

3.7.2 Inventory Database

The current database of CSS structures should be updated to address the missing data and information as discussed in Section 3.1. A sophisticated database should be able to sort and prioritise according to structure type, age, location, material, ratings, inspection schedule and repair priorities. This updated database would be useful to the field inspector, program manager, and maintenance and repair personnel.

As pointed out by Garlich and Thorkildsen (2005), one of the biggest problems in inspection of these structures is the lack of information. Historical records such as as-built plans, maintenance repairs, and installation of new sign panels are important in the management of the structure in the long term, given that there are many changes to the design specifications of sign structures over the years.

As discussed in Section 3.1.4, handover inspection reports are held by the regional asset managers. These reports ensure proper fabrication and construction of the CSS structure. The handover inspection is used to confirm proper alignment, levelling, thread engagement and correct tightening of all connection components. Additionally, this inspection would provide a check that all components such as lock washers, sign clips, U-bolt and other connection hardware are present (Gostautas et al. 2015). Therefore, handover inspection reports of Main Roads CSS structures should be included in the database for ease of access.

It is worth noting that similar to other road agencies, Main Roads is managing inventory of CSS structure by the same methodology and policy that are used for bridges. Two documents that are used for bridges were referred to in Main Roads (2013a), including:

- *Structures Engineering Management System, Part 3 – Procedure for the Management of Bridge Inspections*, document 3912/01/03, for a description of the management process for the inspection, investigation and subsequent recording of maintenance or management requirements for bridges and associated structures.
- *Structures Engineering Management System, Part 4 – Procedure for the Management of Bridge Data & Information*, document 3912/01/04, for the process to be followed in the storage and maintenance of bridge data and information used for the management of the structures asset.

3.7.3 Inspection Types and Frequencies

Table 3.2 summarises the frequencies for Level 2 inspections of CSS structures overseas and in several Australian road agencies. While Main Roads requires a 7-year cycle for all CSS structures, other road agencies commonly require 4–6 years maximum, with TMR as an exception with a 2-year frequency. In addition, the frequency used elsewhere is generally risk-based, which is determined based on a number of factors such as structure condition, age, type and size of the structure.

Main Roads' 7-year frequency is relatively low compared to other road agencies. There was no evidence for the need for reducing it in a short term, given that Main Roads' CSS structures are of a young age. However, the frequency may change in the future when the structures are aging further. A minimum of a 5-year interval is recommended for a structural inspection process, which includes ground-based visual inspection and anchor bolt sounding.

Table 3.2: Comparison of Level 2 inspection frequency for CSS structures

Road agency	Maximum frequency	Notes
Overseas	4–6 years	Reduced frequencies (2–3 years) are applicable to visual, ground-based inspections
Main Roads	7 years	First inspection at 10 years after construction
TfNSW	5 years	2 years for high-risk structures
TMR	2 years	2 years for CS1–2, and one year for CS3–4
DoT	5 years	Depending on structure condition rating. 2 or 3 years are applicable for structures in good condition
DIT	8 years	4–5 years are applicable to structures of old age, long span or located within 1 km of the ocean

It is worth noting that as stated in Main Roads (2013b), the target of a 7-yearly cycle will be subject to review when there is sufficient information available. As there is still limited quantitative data that has been collected and assessed, an optimum frequency for inspection has not been finally determined. The frequency of inspections is influenced by the importance of the bridge or gantry, the complexity of its structural form, durability design, quality of construction, age of the asset and performance history.

3.7.4 Maintenance, Rehabilitation and Renewal

As discussed in Section 3.4, there is no data/information about maintenance, rehabilitation and renewal of CSS structures available for review, because Main Roads CSS structures are of a young age. It is understood that the policies for the maintenance, rehabilitation and renewal of CSS structures are similar to those used for bridges. The following considerations are worth considering to be included in the policies for the management of CSS structures:

- Critical and emergency structural findings should be defined from inspections (VDoT 2014):
  - Critical findings are defined as imminent conditions that could, if left unresolved, result in localised or complete failure (collapse) of the structure, or present a safety issue to the traveling public, and they should be addressed within 90 days of their discovery.
  - Emergency findings are defined as conditions that are deemed to pose an immediate safety risk or hazard to the structure's integrity and/or the traveling public and require immediate attention and corrective action.
  - Common critical findings include pedestals, anchor bolts, grout, poles and base plates, superstructure members and connections, sign connections and attachments, and other components which are in danger of falling.
- Triggerable events for a detailed Level 3 inspection should be defined in the inspection manual (Gostautas et al. 2015). A triggerable event would be defined as an event in which 2 or more major deficiencies were observed, or if the observed deficiencies would result in a condition rating of CS4, or other special or extreme events (e.g., fire, impact, special weather conditions).

4 Risk-based Framework for Management of CSS Structures

4.1 Potential Structural Risks Associated with Main Roads Sign Structures Established Through Trial Risk Assessment Workshop

The previous sections of this report established a number of issues that require a high-level risk review. The key issues that should be focused on include

- the prevalence of long-lever cantilever sign support (CSS) structure failures within Australia and internationally
- the multiple causes for these failures, including issues with design and construction which led to failure mechanisms influenced by fatigue, vibration, and vortex shedding
- the impacts of multiple failure causes and impacts from a fatigue and ultimately a final yield perspective as an accumulation are not collectively known for CSS structures
- the risk to the public driving beneath the signs.

To manage the risk from the component compliance concerns and past performance of similar structures, a management strategy should be developed and implemented to ensure the safety of the travelling public and to mitigate risks for Main Roads and the WA Government. A trial risk assessment was conducted, which included a high-level risk assessment that sought internal and external expertise to offer both advice and independent facilitation. The trial was based on the Main Roads risk management procedure. The risk assessment obtained a collective view from the relevant engineers and asset owners on the potential risks associated with CSS structures taken from a high-level view.

4.1.1 Risk Workshop Preparation

A project launch meeting was conducted at the start of the project to explain the process. A review of logistics planning and coordination of times was established for interviews. A review of Main Roads, ISO and industry standards was conducted as risk assessment process documents to establish and optimise the risk assessment process. Next, CSS risk domains were collected from previous ARRB and Main Roads project research work. Once risk domains were understood from Main Roads and other sources, ARRB developed content for stakeholder communication.

4.1.2 Pre-risk Workshop Interview

Interviews for potential risk domain areas were conducted to assess any new threats emerging or disappearing. The interviews helped establish the risk domains that needed to be included. The risk domains raised from the interviews were:

- CSS collapses onto freeway with safety implications (is the mode of failure going to be ductile enough to change the speed of failure)
- CSS collapses onto freeway causing loss of reputation (where community does not tolerate the failure causes)
- Loss of CSS integrity impacts transport services. Leads to congestion and community impacts. Loss of support for smart highways program, mitigation controls, however, were place. Challenging maintenance access was also discussed.
- Initially risk assess current domains against the CSS. Risk assessment will include inherent, residual and control effectiveness.

All interviewees were co-operative, and insights gained were helpful to establish workshop requirements. The preparation efforts by all reduced time impacts for stakeholders at the online workshop.

4.1.3 Pre-workshop Preparation

Domains and risk technical issues were scoped to provide context for risk assessments. Workshop pre-read was developed for the identified risk domain areas and circulated prior to the workshop. The pre-read is shown in Appendix B.1.

4.1.4 Workshop

An online workshop was conducted on 20 April 2021 to review and align risk view using a combined workshop. Appendix B.1 outlines the workshop outcomes. The workshop was facilitated using the Main Roads internal risk assessment template D11#208987 (Main Roads 2019), with the final results summarised in the Main Roads D11#208987 form used in the workshop. The workshop agenda included:

- project introduction & workshop objectives
- run through Main Roads risk management procedure
- summary of topics covered in pre-meetings
- frame and agree on the issues and consequences for assessment
- assess risk using the Main Roads risk management procedure
- discuss control effectiveness ratings
- risk treatment planning.

4.1.5 Trial Risk Assessment Outcomes

Table 4.1 below presents a summary of the risk assessment outcomes. The full detailed risk assessment is shown in Appendix B.1.

Table 4.1: Summary of Draft Trial Risk Assessment Results

Risk category	Risk description	Inherent risk rating	Residual risk rating	Control rating	Risk review date
Safety	FR1: CSS collapses onto freeway with safety implications (is the mode of failure going to be ductile enough to change the speed of failure)	High	Medium	Adequate (1)	Annual
People, reputation	CSS collapses onto freeway causing loss of reputation (where community does not tolerate the failure causes)	Medium	Low	Adequate (1)	Annual
Service delivery	Loss of CSS integrity impacts transport services. Leads to congestion and community impacts.	Medium	Low	Adequate (1)	Annual
Service delivery	Loss of support for smart highways program, Mitigation controls in place, including challenging maintenance access	Low	Low	Adequate (1)	Annual

1. Control ratings were rated adequate as a median by the workshop attendees. However, maintenance intervention, CSS inventory management, design unknowns regarding fatigue life and inspection controls were classed as inadequate with a number of workshop discussion ideas to improve controls included below.

Source: Main Roads (2019).

4.1.6 Risk Impacts and Causes for Risk Prioritisation Framework

During the workshop, the highest impacts by potential consequences using the Main Roads Risk Procedure were established as safety with the lesser impacts of reputation and loss of road amenity also considered.

The factors selected by the review included:

- materials failure being under specification
- lack of inspection
- under design, in particular for fatigue
- lack of focus and lack of maintenance
- lack of production Inspection Test Plan (ITP) and quality planning rectification during construction
- not knowing structure performance in environment against the design, e.g., wind and traffic
- longer span cantilever design
- environment e.g., salt spray, location of winds
- natural disaster events like high wind.

Vehicle impacts were considered less likely to cause premature failure because of the barrier controls in place.

4.1.7 Control Improvements Discussed at the Workshop

During the workshop, the Main Roads Risk Procedural process recorded concerns raised around the highest impacts by potential consequences. Concerns were:

- The unknown – 'we don't know what we don't know' about fatigue particularly with no Australian Standard for fatigue design.
- Regular overall CSS risk control effectiveness reviews will be needed as control effectiveness could be dynamic.
- Monitoring CSS performance as a unique group rather than lumped together with other asset classes will improve previous asset maintenance and monitoring focus levels.
- Variable Level 1/Level 2/Level 3 monitoring controls – depending on the CSS local environment and budget.

Control improvement suggested at the workshop included:

- Structural Asset Management plan(s) for major structures linked with responsibility matrix.
- Establish a budget to fix, replace and develop timelines for maintenance inspection types based on risk prioritisation.
- Alliance monitoring of baseline performance on CSS is critical to understand in-field performance, acceleration and fatigue stress/strain.
- Establish control effectiveness frequency.
- An assurance standard and program to audit and to check that inspections are occurring is needed to ensure the treatment program is adhered to.
- Damping devices could be considered on structures, particularly when structures are found to have concerns with various wind cyclical loadings.

4.1.8 Trial Risk Assessment Observations

The risk assessment process trial using the Main Roads risk framework was successful because it collated the general technical views of diverse workshop participants into one D11 internal form (Main Roads 2019) using the Main Roads risk management procedure.

The trial found that the internal Main Roads risk procedure can summarise and clarify the technical risks in a way which all participants could understand. A workshop consensus did not take long to achieve using the Main Roads set of tools – primarily due to the teamwork shown by the stakeholders participating in the trial. The trial established that apart from safety, non-technical impacts such as reputation and finance could also be used to prioritise strategic risk for an asset class and may influence potential control budget levels and prioritisation.

Control quality effectiveness and potential action plans were also considered in a measured fashion. The quality of the workshop outcome and time taken was enabled by using pre interviews to focus the workshop agenda and risk domains. It is recommended that this process be used in future to check as part of a control monitoring process to ensure that new emerging issues requiring control changes are not missed and that control effectiveness is recorded and acted upon as necessary.

The trial was successful in looking at all technical risks from a high-level strategic perspective. However, the use of the process would likely be relevant for use on an annual basis. The exception may be if the materiality or criticality of the asset class risk changed – then, the process may be conducted more frequently than annually.

Multiple recommendations emerging from the workshop in support of the framework included:

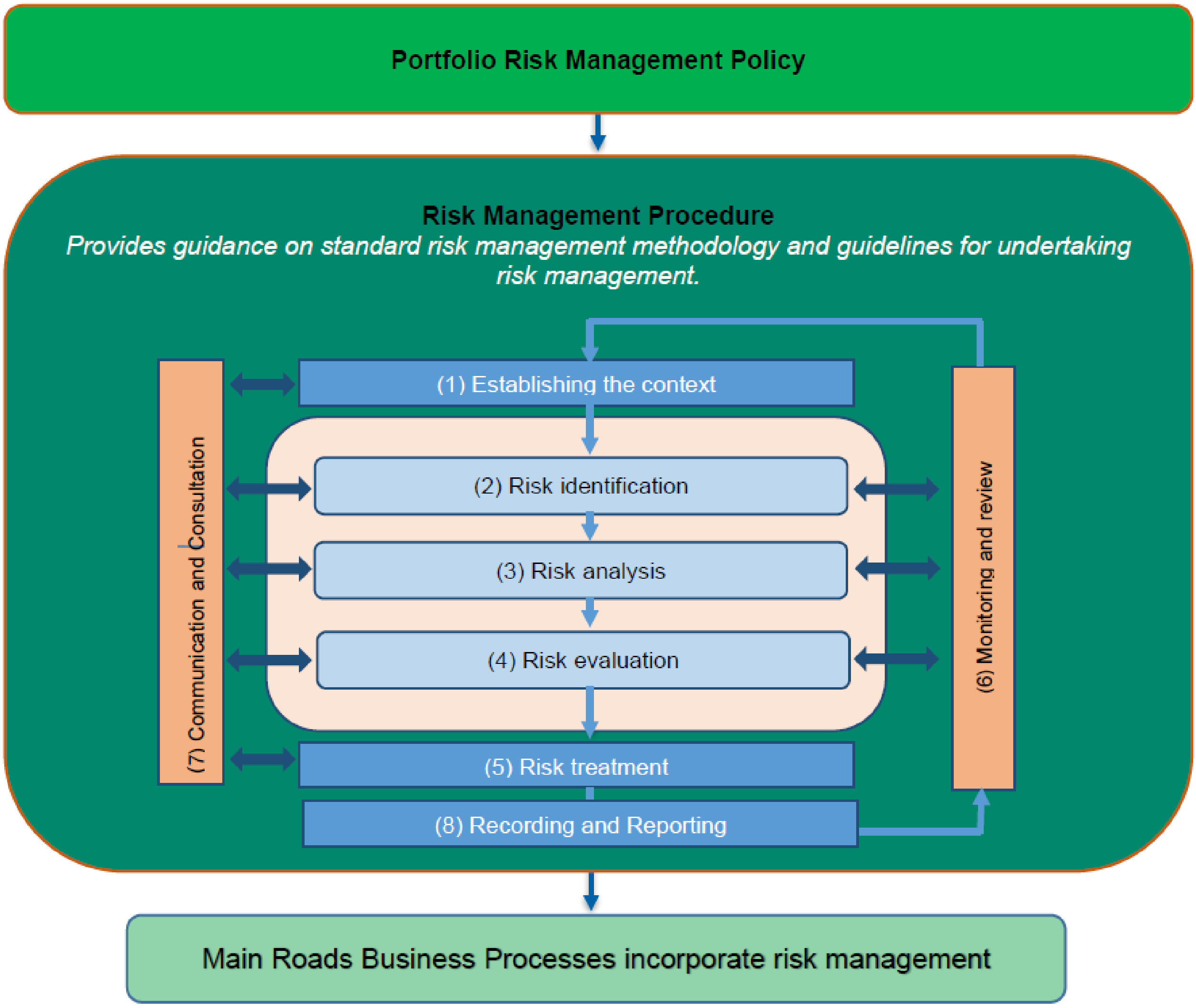
- Consider resolving the inadequate controls identified during the workshop.
- This trial risk assessment process may potentially be used as a part of the future management strategy for CSS structures.
- Adjust risk assessments and control effectiveness scores once the in-field performance baseline of the new CSS assets is understood.
- Structural Asset Management plan(s) and strategies for major structures are important and are currently being developed for the CSS asset group. The Main Roads' Alliance are providing these, with a workshop recommendation to consider linking plans with the responsibility matrix.
- Establish a budget to revise the CSS life-cycle timelines, inspection timelines by inspection type.
- Alliance Monitoring of baseline to understand in-field performance, acceleration and fatigue stress/strain (future waiting on the proposal).
- Consider developing frequencies of future control effectiveness checks.
- Develop an audit program to confirm the treatment program is being adhered to.
- Consider vibration monitoring or cyclical monitoring in a sensible method to reduce reliance on standard life span assumptions.
- Consider the use of damping devices on CSS structures when structures have concerns with winds with a caveat that this would be subject to understanding the baseline performance loading on the structure relative to the design and actual limits.
- Consider low stress, cyclical monitoring of critical CSS structures to establish fatigue criticality before deciding on costly CSS early replacement or inspection frequencies in future asset management strategies.

4.2 The Proposed Framework

The objective of developing a risk-based approach is to work within the Main Roads framework and consider asset class–specific technical risks to allow the allocation of limited resources using a rational basis for prioritisation. The primary reason for proposing this risk-based framework for managing the CSS assets is to improve budgeting and asset management decisions from a technical position. The primary output of the risk-based approach proposed is the ability to make informed budget allocation decisions with an understanding of the associated risk(s).

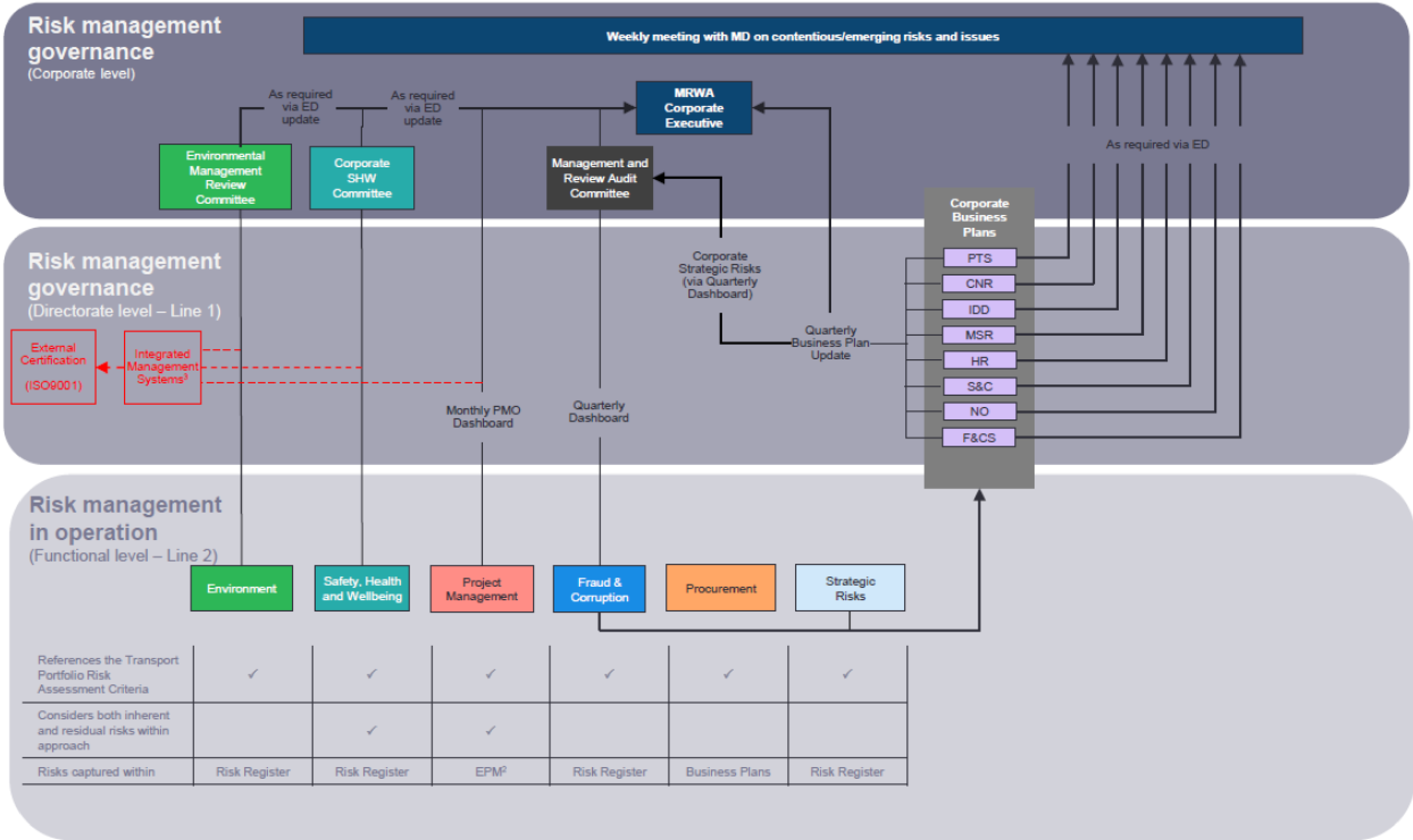
The risk-based framework for managing CSS structures must be integrated with the defined levels of service for asset management plans. This will define the impact of a CSS structure's condition on levels of service duty and other factors. The framework for CSS risk-based assessment is based on the Main Roads Risk Management Procedure and operating model requirements as shown in Figure 4.1 and Figure 4.2 below. According to this framework, the risk management process is embedded in existing processes where possible. In this approach, risk management is not viewed as a separate discrete activity but forms an inherent part of business processes where everyone is involved in the management of risk.

Figure 4.1: Main Roads Risk Management Procedure



Source: Main Roads (2019).

Figure 4.2: Main Roads Risk Operating Model



Source: Main Roads (2019).

The trial risk workshop established higher inherent and residual risk levels involving the safety of motorists travelling beneath signs. Other impacts such as reputation and sign amenity were lower based on the workshop consensus view. The following Table 4.2 outlines how Safety, Health and Wellbeing (SHW) risks are identified and managed within Main Roads. These requirements are set by the Safety, Health and Wellbeing branch (Safety Branch).

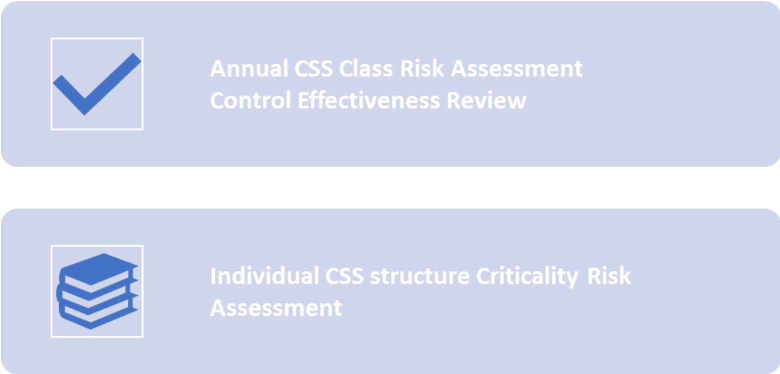
Table 4.2: Main Roads SHW risk identification process

Step	Application
1. Establish the context	SHW risks are identified via annual risk workshops: • Directorate and location workshops (including Main Roads' regions); and • Organisation-wide risk assessments undertaken by the Safety Branch and Safety Resources from across Main Roads.
2. Risk identification	SHW risks are identified.
3. Risk analysis	Each SHW risk is given a residual rating based on the consequence and likelihood of each risk.
4. Evaluation	The adequacy of the existing controls is considered to determine what risk treatment actions are needed.
5. Risk treatment	• Risk treatment actions for High and Very High risks from the directorate and location workshops are included in the SHW Improvement Plans. • Risk treatment actions for High and Very High risks from the organisation-wide risk assessments undertaken by the Safety Branch and Safety Resources from across Main Roads are included in the 3-year Safety Strategy.
6. Monitoring and reviewing	SHW risks are reviewed annually or if there has been a significant incident or a change to work structure. 1. Reporting on risks – Local level: Risks identified at the directorate and location risk workshops are captured in the SHW Risk Register, which is reported to the directorate and local Safety Committees annually. All High and Very High risks from directorates and locations are also included in the Corporate Risk Register. – Corporate level: All High and Very High risks from the organisation-wide risk assessment are recorded in the Corporate Risk Register and are reported to the Corporate SHW Committee annually. 2. Reporting on Risk Treatment Actions – Local level: The completion status of actions in the SHW Improvement Plans are reported to the directorate and local Safety Committee. – Corporate level: The completion status of actions in the 3-year Safety Strategy are reported to the Corporate SHW Committee every 6 months (June and December).

Source: Main Roads (2019).

The process proposed includes details from the trial workshop, literature review performed for this project and an investigation into asset risk prioritisation utilised in a number of sectors including mining, construction and transport. It is proposed that a risk assessment of CSS structures and CSS risk management should occur for both the complete asset class and for the individual CSS structures in 2 separate processes, Class and Individual, as shown in Figure 4.3.

Figure 4.3: Two levels of CSS risk assessment: Class and Individual



4.2.1 Annual CSS Class Risk Assessment and Control Effectiveness Review

The annual CSS class risk assessment proposed includes using the Main Roads risk procedure and template example shown in Main Roads (2019) and, in particular, following the risk management procedure Table 2: Main Roads Safety, Health and Wellbeing (SHW) risk identification process elements of:

1. establish the context
2. risk identification
3. risk analysis
4. evaluation
5. risk treatment
6. monitoring and reviewing.

Once the annual risk review is completed, the preparation of a risk appetite statement is needed to inform leadership of the risk position for the CSS structural assets as a group of assets. Strategic sample questions posed by the annual CSS class review could include:

- How do the overall set of control qualities work?
- What controls are missing from the controls in place?
- What is the greatest impact of any failure?
- Assess which controls are critical for budgeting, and those that are less so?
- Is there a need to establish special performance projects like monitoring one CSS structure baseline performance for 5 years?
- Does the criticality assessment and prioritisation process require improvement when opportunities external to the organisation, standards or factors change?
- Is the risk described within the risk appetite statement reasonable? Can we accept this risk? Or does the risk treatment plan require improvement?
- Are the risk controls proposed acceptable and the risk tolerable, if not, what intervention is required?
- Do the Trigger and Response intervention plans need changing? (e.g., intervention maintenance actions or traffic deviation thresholds correct for the revised asset level prioritisation).

Any high residual risk should be elevated to the corporate Enterprise Risk Register (ERR) as per the Main Roads risk management operating model. In addition, on an annual basis, it is proposed that the CSS criticality framework be assessed and adjusted to improve the system as an increased understanding of CSS structural performance is achieved.

4.2.2 Individual CSS Structure Criticality Risk Assessment

The individual CSS structure criticality risk assessment performed for each asset in the CSS register will require the addition of individual CSS asset structural hazard assessments including the review of inspection reports and an accurate CSS register of asset details. This process uses the Main Roads risk procedure and template example shown in Main Roads (2019) and follows the risk management procedure Table 2: Main Roads SHW risk identification steps:

1. establish the context
2. risk identification
3. risk analysis
4. evaluation
5. risk treatment
6. monitoring and reviewing.

The individual CSS assessment requires adding a hazard scoring rating for each asset into step (4) and professional judgement should be used to rate the specific risk factors by using the tables below. This score result will be added to the Main Roads D11 template (Main Roads 2019). The hazard risk factors to be selected are presented in Table 4.3 to Table 4.9.

Table 4.3: Network criticality score

Score	Impact on Main Roads network and loss of service
1	Minimal and loss of asset has no impact
2	Moderate
3	High
4	Essential

Table 4.4: Vulnerability scale

Score	Description of response to DVR[1], Level 1 or 2 assessments
1	Not affected (no loss of function)
2	Minor (slight loss of function)
3	Moderate (some substantive loss of function)
4	Complete loss of CSS functionality

Table 4.5: Proximity to environmental factors (wind)

Score	Description of environmental wind factors vs design assumptions (galloping, vortex shedding, truck wind, cyclone, …)
1	Not affected
2	Minor
3	Moderate
4	Major

[1] DVR: Digital Video Road – a tool that allows high resolution Hawkeye video data, collected from road inspection vehicles, to be viewed on a personal computer.

Table 4.6: Age timescale relative to 50-year life span

Score	Age timescale relative to 50 year life span
1	< 10 years old
2	< 20 years old but greater than 10
3	< 30 years old but greater than 20
4	> 40 years old and replacement or repair plan should be in place

Table 4.7: Non-conforming design

Score	Non-conforming design
1	Complies with relevant Standards
2	Minor variations to Standards but still complies fully with fatigue requirements
3	Moderate variations to design Standards
4	Serious variation to design Standards

Table 4.8: Non-conforming materials

Score	Non-conforming materials
1	Complies with relevant materials Standards
2	Minor variations to Standards but still complies fully with design and fatigue requirements or test results confirm similar properties to relevant material Standards
3	Moderate variations to material Standards
4	Serious variation to material Standards

Table 4.9: Inspection results

Score	Non-conforming inspection results
1	Complies with relevant inspection Standards
2	Minor variations to Standards but still minor impact expected within replacement life span recorded in the asset strategy
3	Moderate variations to Standards
4	Serious variation to Standards

Once the scores are selected using professional judgment, the criticality factor is calculated using Equation 1. The highest number corresponds to the highest priority for repair, monitoring or replacement. The highest number is 16,384 and the lowest number is 1 (normalised in the calculation sheet):

$$CSS_{criticality} = \text{Network criticality} \times \text{Vulnerability scale} \times \text{Proximity Env wind} \times \text{Age Timescale} \times \text{Non-Conforming Design} \times \text{Non-Conforming Materials} \times \text{Inspection Results} \quad (1)$$

4.2.3 Informing the Strategic Maintenance Plan for Each CSS Asset

Once a criticality assessment is performed and the priority list is developed for replacement, monitoring and repair, it is important to update both planning and asset strategic plans. In addition, the strategic and maintenance plan updates can be informed using the sample questions below:

- What are the key performance measures for the revised maintenance and monitoring program with the new prioritisation list?

- What are the revised KPIs for each asset for future monitoring or does it stay the same?
- What are the most critical assets to replace?
- Which controls are critical?
- How often will critical controls need to be checked for effectiveness?
- What are the most critical assets to monitor and assess for repair?
- What is the new maintenance inspection frequency for each asset?
- Will the criticality levels and prioritisation need to be re-adjusted and at what frequency?
- What is the cost-benefit value of replacement versus repair and inspect?
- What budget is required to monitoring the critical CSS assets within each environment and condition context?
- Does the current 50-year life span require re-assessing for replacement strategy?
- What budget is required to meet the revised prioritisation?
- Are there any issues that require elevation to the annual risk assessment and governance review?

5 Conclusions and Recommendations

A number of lessons have been learnt during this project. Particularly, during the project phases of literature review, the trial risk workshop and management plan tasks.

5.1 Lessons from the Literature Review

The key learnings regarding the current practices for managing CSS structures are summarised as follows:

- Australian road agencies currently have a more stringent inspection regime for CSS structures than in the USA, with more frequent regular visual inspections (Level 1 and Level 2). Main Roads inspection levels for CSS are lower than some Australian counterparts.
- Most road agencies implement an inspection regime that is similar to that applied to road structures, with regular visual inspection being the most common and supplemented by in-depth, detailed inspections and NDT methods. Given that fatigue defects occur frequently to the base components of the CSS structures, additional inspections are also carried out by some road agencies which focus on these components.
- The main cause of failure in Australia and overseas is fatigue. Fatigue in CSS structures is caused by wind-induced vibrations due to 4 wind loading phenomena including natural wind gusts, galloping, truck-induced wind gusts and vortex shedding. Fatigue defects in CSS structures usually occur at the connection of the mast arm to the pole, at the base of the pole, top of stiffeners, perimeter of hand holes and anchor rods.
- Risk management of CSS structures has been the focus of some road agencies such as DoT Victoria through a regular inspection regime and measures to mitigate potential risks of collapse and additional risks to third party road users.
- CSS structures are susceptible to wind-induced vibrations with minimal damping.
- Short-term monitoring of CSS structures for fatigue defects is not an effective means of managing the risks of failure.

5.2 Lessons Emerging from the Trial Risk Workshop

The pilot risk assessment of the complete asset class of CSS using the Main Roads risk framework was successful and can be repeated annually to align and collate the general technical views of diverse workshop participants into one D11 internal form using the Main Roads risk management procedure. As more is understood from both technical and risk perspectives, the frequency of these reviews could be reduced. The workshop established that some controls were ineffective. However, a much deeper technical review is required for individual CSS structure prioritisation when it comes to risk assessment from a criticality perspective. The pilot established that apart from safety, non-technical impacts such as reputation and finance could also be used to prioritise strategic risk for an asset class and may influence potential control budgeting and prioritisation.

Control quality effectiveness was rated, and potential action plans were established based on detailed pre-workshop interviews. The interviews coalesced the workshop details and risk domain information to aid in reaching a consensus. The process can be used in future to improve the focus in technical asset management, control effectiveness of the controls and to improve the system asset criticality prioritisation. The pilot established a need to link the asset class to the enterprise register should residual risk go over the limits defined by the risk tables within the Main Roads Risk Procedure. The risk workshop confirmed the learnings from the literature review with respect to the need to focus efforts on assessing and monitoring long-term structural performance, including relevant governance oversight to ensure the controls are effective.

5.3 Risk-Based Framework for Management of CSS Structures

During the framework development the learnings were:

- The need to build 2 levels of governance is essential to prevent losing sight of the strategic risk outcomes outside of safety.
- The high level of uncertainty and dynamic nature of CSS technical risks requires 2 levels of risk review to ensure risk controls are being reviewed regularly.
- Due to high levels of uncertainty, professional judgement is required to select the critical risk factors for prioritisation of inspection, testing, repairs and asset replacement.

The risk of apparent greatest concern occurs in the scenario where there is a catastrophic collapse of a CSS that does not coincide with a major storm event or similar. This is most likely an incremental damage scenario, where repeated 'normal' loading results in structural failure. An understanding of these events will be enhanced by capturing representative data, which is sparse both in Australia and internationally. This will help to address 2 current expertise limitations:

1. Quantification of the response of CSS structures to typical daily excitation and the corresponding effects that these have on typical structures.
2. The limit states design/assessment approach, which is typically focussed on the response to extreme events. While fatigue is one of the potential limit states, a meaningful knowledge of it requires an understanding of (1) above.

The proposed medium-term monitoring program over 5 years both demonstrates due diligence and is likely to be addressing the above expertise limitations. Due to high levels of uncertainty, a higher level of focus on control effectiveness checking and inspection is required until asset base performance and a 5-year testing program is completed.

5.4 Recommendations

It is recommended that Main Roads:

- implements the 2 levels of risk review – annual strategic risk assessment for asset class and specific structural asset risk criticality in line with Main Roads risk management operating model
- establishes a second line assurance process to ensure monitoring and inspections occur
- reviews risk control effectiveness regularly according to the monitoring outcomes
- plans sufficient budget for monitoring CSS in line with or better than a similar critical risk assessment process proposed to critically prioritise inspection and maintenance
- establishes a monitoring program for CSS structures in line with report findings and establishes a phased approach for implementing increased due diligence across the asset class. The 2 phased approach will take 3–5 years, with the highest of intensity of work being in year one to finalise criticality and build a database of knowledge on the existing assets. The phased approach recommended will include:

Phase 1 (Year 1)

1. Investigate and establish current CSS asset base using DVR video, update CSS asset register.
2. Implement risk-based framework and categorisation of CSS asset base into integrated risk management systems at Main Roads.
3. Assess DRV road inspection videos. If unavailable – develop and video, then complete the process.
4. Prioritise Level 1 inspection from the DVR assessment and revised criticality assessment.
5. Perform Level 2 inspection plan and monitoring against criticality scores.
6. Level 3 inspections decide high level inspection need versus automatic replacement planning.

7. Update the Main Roads Inspection guidelines.
8. Establish fingerprint benefit-cost ratio costing for accelerometer/strain gauge trial Phase 2.

Phase 2 (Year 2 onward)

1. Establish accelerometer/strain gauges test for CSS structures with no redundant load paths and asymmetrical loading and test over a 5-year period.
2. Implement annual monitoring service by internal or third-party provider to trigger response and develop a basis for understanding.

References

AASHTO 2013, *Standard specifications for structural supports for highway signs, luminaires, and traffic signals*, American Association of State Highway Transportation Officials, Washington, DC, USA.

AASHTO 2015, *LRFD specifications for structural supports for highway signs, luminaires, and traffic signals*, LRFDLTS-1, American Association of State Highway Transportation Officials, Washington, DC, USA.

Bicilli, V 2015, 'Management and maintenance of sign bridges, cantilever sign structures and mast arm structures on French national roads network', *25th World road congress*, *Seoul, South Korea*, World Road Association (PIARC), Paris, France, 9 pp.

Consolazio, GR, Dexter, RJ & Johns, KW 1998, *Fatigue performance of variable message sign and luminaire support structures: volume 1: fatigue related wind loads on highway support structures*, New Jersey Department of Transportation & FHWA, Washington, DC, USA.

Cook, RA & Halcovage KM 2007, *Anchor embedment requirements for signal/sign structures*, Report No. BD545 RPWO #54, University of Florida, FL.

Dexter, RJ & Ricker, MJ 2002, *Fatigue-resistant design of cantilevered signal, sign, and light supports*, NCHRP report 469, TRB, Washington, DC, USA.

Epp, TA, Amorim, DRC & Rihal, SS 2019, 'Discussion on the development and jurisdictional implementation of overhead sign support structure designs and codes', *Transportation Association of Canada and ITS Canada 2019 joint conference and exhibition*, TAC, Ottawa, ON, Canada.

Florea, MJ, Manuel, L, Frank, KH & Wood, SL 2007, *Field tests and analytical studies of the dynamic behavior and the onset of galloping in traffic signal structures*, FHWA/TX-08/0-4586-1, Center for Transportation Research, University of Texas & Federal Highway Administration, Washington, DC, USA.

Fouad, FH, Davidson, JS, Delatte, N, Calvert, EA, Chen, SE, Nunez, E & Abdalla, R 2003, *Structural supports for highway signs, luminaires, and traffic signals,* NCHRP report 494, TRB, Washington, DC, USA.

Gallow, MS, Fouad, FH & Hosch IE 2015, *Mitigating fatigue in cantilevered overhead sign structures*, Transportation Research Record: Journal of the Transportation Research Board, No. 2522, Transportation Research Board, Washington, D.C., 2015, pp. 18–26.

Garlich, MJ & Thorkildsen, ET 2005, *Guidelines for the installation, inspection, maintenance and repair of structural supports for highway signs, luminaires, and traffic signals*, FHWA NHI 05-036, Federal Highway Administration, Washington, DC, USA.

Ghaedi, H, Nims, D, Gostautas, R, Steinberg, E, Hu, L, & Walsh, K 2016, *Field study of Ohio's structural support inspection program for overhead signs, traffic signals, and high-mast lights*, Transportation Research Record: Journal of the Transportation Research Board, No. 2550, Transportation Research Board, Washington, D.C., 2016, pp. 15–21.

Gostautas, R, Nims, D, Steinberg, E, Hu, L & Walsh, K 2015, *Evaluation of overhead support inspection program,* MISTRAS Group, Ohio Department of Transportation & Federal Highway Administration, Washington, DC, USA.

Hosch, IE, Fouad, FH, Gallow, M & Traylor, L 2017, *Operational post deformation of cantilever sign support structures from truck-induced wind gusts*, Pract. Period. Struct. Des. Constr., 2018, 23(1): 04017033.

Kaczinski, MR, Dexter, RJ & Van Dien, JP 1998, *Fatigue-resistant design of cantilevered signal, sign, and light supports*, NCHRP report 412, TRB, Washington, DC, USA.

Morris, D & Thomas, M 2009, 'In-service monitoring of a cantilever sign gantry', Austroads Bridge Conference, 7th, 2009, Auckland, New Zealand.

Nims, DK, Guner S & Lawrence, S 2019, *Overhead Sign Failure Investigation*, Task 1 Final Report, FHWA/OH-2019-11, Ohio Department of Transportation, OH.

Rowe, JB 2005, 'Taking overhead sign structure inspection to the next level: element-based overhead sign structure inspections', Paper 06-2803, 86th Annual Meeting, TRB, Washington, DC.

Virginia Department of Transportation 2014, *Traffic ancillary structures inventory & inspection manual,* VDoT, Richmond, VA, USA.

Australian Road Agencies Technical Guidelines

Austroads 2021, *Design and construction guidelines for the delivery of large cantilever and gantry structures*, AP-G95-21, Austroads, Sydney, NSW.

Department of Planning, Transport and Infrastructure 2020, *Roads structures inspection manual*, RAMA-ST-PRC-048, DPTI, Adelaide, SA.

Main Roads Western Australia 2013a, *Sign gantry guidelines (level 1 and level 2 inspections),* document no. 6706-02-2239, MRWA, Perth, WA.

Main Roads Western Australia 2013b, *Structures inspection and information management policy*, document no. 6706-01-202, MRWA, Perth, WA.

Main Roads Western Australia 2015, *Cantilever signs: sign structural design guidelines*, document no. 67-08-4K, revision 2G, MRWA, Perth, WA.

Main Roads Western Australia 2016, *Type B and C VMS gantry concept design: usage and design information*, document no. D16#380856, MRWA, Perth, WA.

Main Roads Western Australia 2019, *Risk management procedure 2019*, document no. D11#208987, MRWA, Perth, WA.

NZ Transport Agency 2020, *Specification for permanent traffic signs: specification for the design, manufacture, installation and maintenance of permanent traffic signs*, P24:2020, NZTA, Wellington, NZ.

Queensland Department of Transport and Main Roads 2020, *Structures inspection manual*, TMR, Brisbane, Qld.

Queensland Department of Transport and Main Roads 2021, *Design criteria for bridges and other structures: part D: other structures, chapter 7: gantries and support structures for signage, lighting and ITS*, TMR, Brisbane, Qld.

Queensland Department of Transport and Main Roads 2014, *TMR Standard Drawing 1581 ITS Cantilever: Cantilever Structure*, TMR, Brisbane, Qld.

Roads and Maritime Services 2013, *Processes and procedures manual for structural integrity inspection and condition assessment of traffic asset structures*, RMS, Sydney, NSW.

VicRoads 2018, *Road structures inspection manual*, VicRoads, Kew, Vic.

VicRoads 2019a, *Sign gantries and lighting masts*, Code of Practice, BTN 014, VicRoads, Kew, Vic.

VicRoads 2019b, *Guidelines for the protection of gantry and cantilever sign supports*, Road Design Note, RDN 06-13, VicRoads, Kew, Vic.

Standards Australia

AS/NZS 1163: 2016, *Cold-formed structural steel hollow sections.*

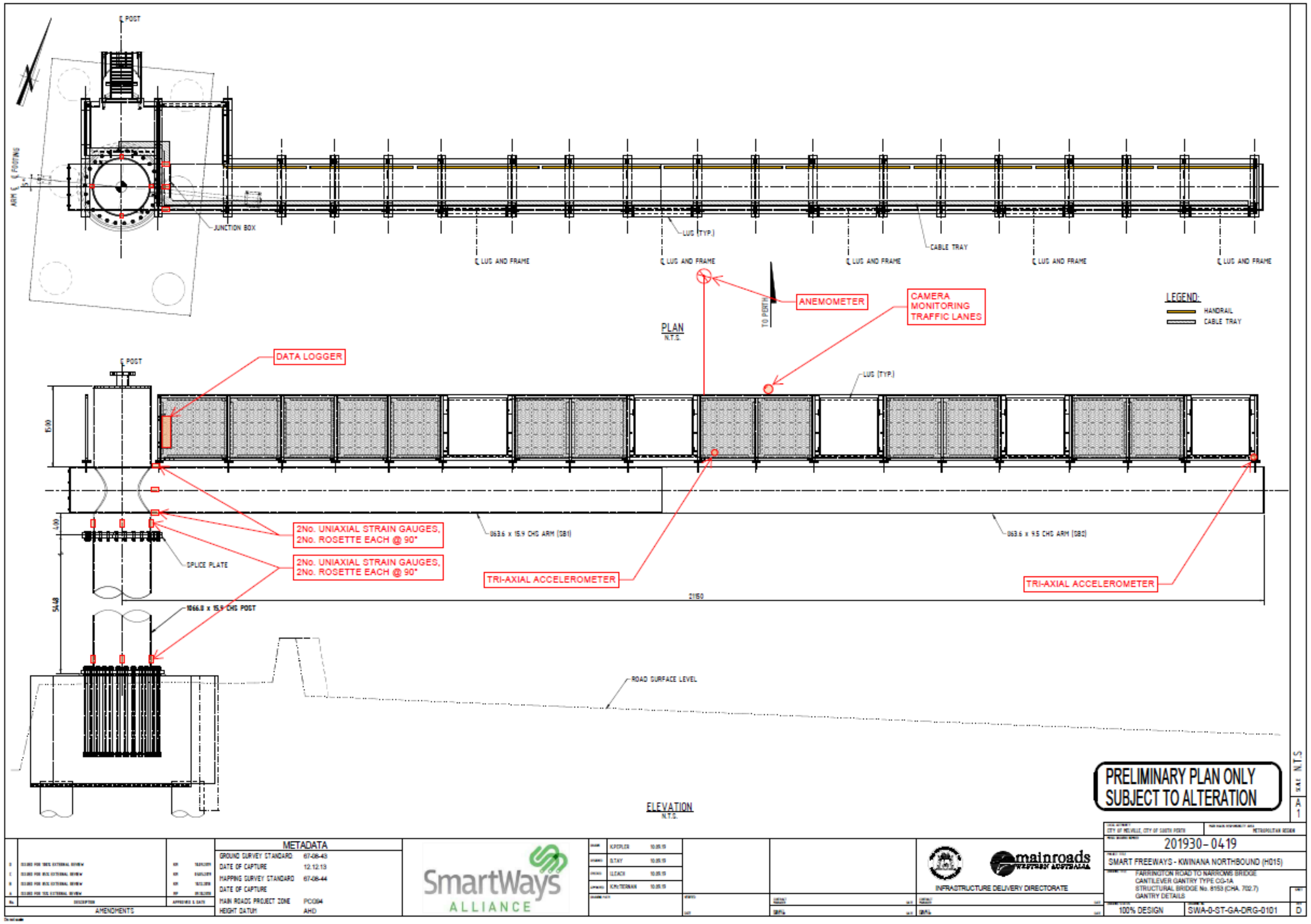
AS 4100: 2020, *Steel structures.*

AS 4291.1: 2015, *Mechanical properties of fasteners made of carbon steel and alloy steel Bolts, screws and studs.*

AS 5100.2:2017, *Bridge design: part 2: design loads*.

AS/NZS 5100.6:2017, *Bridge design: part 6: steel and composite construction*.

Appendix A Current Main Roads Monitoring Proposal



Source: Provided by Main Roads.

Appendix B Risk Workshop

B.1 Pre-read Documentation

Risk workshop brief: Broad-Brush Risk Assessment (BBRA) of MRWA long-lever cantilever sign support structures

Background

Simon Orton of SOA (https://soadvisory.com/) has over 25 years' experience in engineering risk management and has assisted construction, government utilities and mining boards report on risk profile and clearly understands the appetite and tolerance for risk. In addition, Simon has worked in the past to develop and implement controls in fatigue for dynamically moving structural steel systems.

Over the last 12 months, SOA has refined a BBRA process specifically for the COVID-19 environment. This BBRA process reduces the need to travel to site for the BBRA assessment. This includes the use of online interview/workshop meetings (Microsoft Teams or Zoom) and reports that build a summary picture of each risk domain. The resulting report(s) have been accepted well by various Councils, Government departments, Australian Company executives and Board members wishing to clearly outline and understand the risk issues. This process is particularly useful to those working remotely and unable to participate in person for the BBRA processes. BBRA facilitation provides an agreed collective internal view of the risk appetite and does not provide third party judgement of risk or control effectiveness.

Introduction

Over the last two decades, a number of long-lever cantilever sign support (CSS) structures have failed within Australia and overseas. There have been numerous causes for these failures which include issues with design and construction which led to failure mechanisms like fatigue, vibration and vortex shedding. Recent studies indicate that higher stresses and a greater number of fatigue cycles than were previously anticipated are caused by wind and upward pressure from large vehicles passing beneath the structures. While there have been improvements in the Australian Standards and Main Roads Technical Standards based on the learning from these past events, the overall safety and long-term performance have not been assessed/verified.

Furthermore, concern has been raised at the compliance of these structures to the relevant Australian Standards as the components could not be sourced within Australia and relevant certification could not be obtained to verify if they comply with Main Roads requirements.

To manage the risk from the component compliance concerns and past performance of similar structures, a management strategy should be developed and implemented to ensure the safety of the travelling public and mitigate technical risks for Main Roads and the WA Government. There is a strong need to establish a knowledge base with respect to the performance of and potential risks associated with long-lever cantilever sign structures, in order to develop a methodology to monitor and proactively manage the risks associated with these structures.

As part of a WARRIP project 2020-016 Management of Long-lever Cantilever Sign Structures, ARRB has reviewed and consolidated current practice in the technical issues and management of CSS structures. Three key risk domains have been identified, including material uncertainties, structural defects (such as fatigue, fractures and other failures experience across Australia), and site-specific condition (wind loads). It is intended that a risk assessment be conducted to obtain a collective view from the relevant engineers and asset owners on the potential technical risks associated with CSS structures.

Purpose

The purpose of this program is to conduct a low impact trial Broad-Brush Risk Assessment (BBRA) baseline risk review with key senior stakeholders using the MRWA Risk framework. This will be a minimal impact, remote set of interview engagements to establish a contiguous risk view across stakeholders. This project will provide input to the development of a risk-based methodology to manage technical risks associated with CSS structures.

Engagement program

All the interviews and BBRA sessions are proposed to be run across 2 hour blocks each day from 1 – 3pm UTC +8 (Perth time). The scope proposed includes:

Establish BBRA baseline enterprise risks for MRWA CSS structures

The Broad-Brush Risk Assessment process involves the following scope elements:

1.1 Foundation preparation

- Kick off Zoom or Teams meeting at start of project to explain the process (30 mins)
- Logistics and coordination
- Review current risk registers at enterprise level
- Review RFI documents
- Collate CSS risk domains from previous ARRB and MRWA project work
- Establish and estimate likelihood from MRWA and other events
- Develop tool for BBRA assessment

1.2 Interviews for potential risk domain areas

- Assess any new threats emerging or disappearing
- Agree risk domains that need to be included (e.g., based on recent CSS incidents, environment change or new threats)
- Review past CSS risks on the MRWA enterprise register and see if any changes have occurred
- Initially risk assess current domains against the sign support structures (CSS). Risk assessment will include inherent, residual and control effectiveness

1.3 Write the identified risk domain areas and circulate for workshop pre read

- Word Document summarising the identified risk domain areas
- Circulate report for comments and changes
- Send final preparation report for workshop pre-read

2.0 Online workshop to align risk view in a combined Teams workshop

3.0 Finalise basic BBRA report & tool based on an aligned position

ARRB Resources

BBRA facilitator Simon Orton will provide the facilitation with extensive experience in BBRA facilitation.

MRWA Resources

For a successful project delivery – the following resources and low impact support will be required from MRWA please:

- Contact details and access to appropriate senior personnel across for short blocks of time for interviews and the workshop
- Access to existing documents and risk registers at enterprise and regional for the structures. MRWA resources will be required to give access to relevant case studies and events information so the risk

domains can be carefully crafted prior to the interviews. Where possible this support will come directly, internally from ARRB with an extensive history of understanding the domain

Approach and Timing

- Project can commence in late March 2021
- Project delivery will take approximately 3 to 6 weeks – depending on stakeholder availability.

A phased approach for the risk assessments is recommended.

Phase	Description	Proposed Dates*
1.1	**Foundation development (Preparation)**	15 to 21 March
1.2	**Interview dates**	22 to 26 March (1-3 pm UTC+8)
1.3	**Pre read report date**	29 March
2.0	**Workshop and risk assessment** Document risks (e.g., risk owner, ratings, controls, etc) Setup a BBRA and register of risks Facilitate workshop by remote to agree risk and control review ratings	Wednesday 31 March 2021 1pm (UTC +8)
3.0	**Provide final risk assessment report**	10 April 2021

B.2 Trial Risk Workshop Outcomes, D11#208987

CSS Risk Trial Workbook

PLEASE Note this file is not use outside of this trial assessment

Process

Based on the key objectives, guiding principles and role of the [Directorate/Branch] identified as part of the [Directorate/Branch] business plan, workshop attendees were requested to identify risks that would prevent the achievement of these objectives. The causes and consequences contributing to each key risk were also identified, collected and grouped into respective themes. The risks were rated in terms of their likelihood and consequence.

Instructions

This workbook presents the key safety, reputation and infrastructure integrity risks identified (and their risk ratings) with hyperlinks to any identified supporting causes and consequences and their respective treatment plans. Further detail for these causes and consequences are available through the links on each page.

See the Key Risks

Main Roads WA

Draft trial CSS Risk Workbook

Workshop held on 21 April 2021

To access supporting detail (i.e. Consequences and causes for each risk) click on the respective link in column
Treatment planning was considered for risks rated high and above only.
Go to Heatmap
Back to Instructions

RISK MATRIX					
LIKELIHOOD	CONSEQUENCE				
	1 Insignificant	2 Minor	3 Moderate	4 Major	5 Catastrophic
5 Almost Certain:	L	H	H	VH	VH
4 Likely:	L	M	H	VH	VH
3 Possible:	L	L	M	H	H
2 Unlikely:	L	L	L	M	H
1 Rare:	L	L	L	L	M

[Note: Delete columns D-G if only the residual risk is being rated, not the inherent risk]

Key Risks

Risk Category	Risk ID	Risk Description (Risk Title)	Consequence	Likelihood	Inherent Risk Rating		Consequence	Likelihood	Residual Risk Rating		Risk Owner	Control(s) require improvement? (Yes/No)	Business Plan or Treatment Action reference	Risk Review Date
Safety	FR1	CSS collapses onto freeway with safety implications (is the mode of failure going to be ductile enough to change the speed of failure)	Catastrophic (5)	Unlikely (2)	10	High	Catastrophic (5)	Rare (1)	Medium	Medium	Asset Manager Structures	Yes	Trial Only	Yearly reduced to couple of years later
People, Reputation	FR2	CSS collapses onto freeway causing loss of reputation (where community does not tolerate the failure causes)	Major (4)	Unlikely (2)	8	Medium	Major (4)	Rare (1)	Low	Low	Asset Manager Structures	Yes	Trial Only	Yearly reduced to couple of years later
Service Delivery	FR3	Loss of CSS integrity impacts transport services. Leads to congestion and community impacts.	Major (4)	Unlikely (2)	8	Medium	Major (4)	Rare (1)	Low	Low	Asset Manager Structures	Yes	Trial Only	Yearly reduced to couple of years later
Service Delivery	FR4	Loss of support for smart highways program, Mitigation controls in place, Challenging maintenance access	Minor (2)	Unlikely (2)	4	Low	Minor (2)	Rare (1)	Low	Low	Asset Manager Structures	Yes	Trial Only	Yearly reduced to couple of years later
	FR5													
	FR6													
	FR7													
	FR8													
	FR9													
	FR10													

FR1 - CSS collapses onto freeway with safety implications (is the mode of failure going to be ductile enough to change the speed of failure)

Risk Owner: Catastrophic (5)

Return to Key Risks
Back to Heatmap
Treatment Plan

Risk Category

Safety

Causes

Safety
Materials failure /under specification
Lack of inspection
Lack of focus and lack of maintenance
Lack of production ITP and quality plan during construction
Standards need improving eg no Aust std
Under designed
Long span cantilever design
Treat the asset like a sign and not a bridge asset
Environment eg. Salt spray
Vehicle Impacts (Unlikely with controls in place)
Natural disaster events

Consequences

Injury
Fatality

Key Controls	Control adequate? Select from below	Note(s)
Design		The unknown - "we don't know what we don't know" - add to treatment plan the investigation and recheck of the rating
Inspection L1, L2, L3?	Requires Improvement	Procedures adequate but budget is not adequate and full round has not been completed on first round - some difference of opinion - most conservative rating chosen
External Affairs process and ministerial and strategic & communication directorate	Adequate	
Verification of designs	Adequate	Variable L1/L2/L3 monitoring controls - depending on the CSS local environment will be needed
Construction testing and requirements	Adequate	Material testing quality has varied and past ITP during construction has been resolved
Maintenance and intervention in a timely manner from inspection reports	Inadequate	Funding and resources are the drivers
System of recording maintenance control include inventory records	Inadequate	Inventory does distinguish between gantries and cantilevers - use off line unsustainable processes and not updated yet.
New Control		Monitoring CSS performance as a unique group
Structural Asset Management plan(s) for major structures (on its way for CSS from the Alliance)		
Establish a budget to fix?		
Future control - Asset replacement planning , eg 30 years not 100 years structural design life and sign 50 years		Replacement cost can be lower than investigation and repair sometimes
Alliance Monitoring of baseline to understand infield performance, acceleration and fatigue stress/strain (Future waiting on the proposal)		$Regular overall CSS risk control effectiveness reviews will be needed

Effectiveness of controls

Adequate

Risk Review Date

21-Apr-21

FR2 - CSS collapses onto freeway causing loss of reputation (where community does not tolerate the failure causes)

Risk Owner: Major (4)

Return to Key Risks
Back to Heatmap
Treatment Plan

Risk Category

People, Reputation

Causes

People, Reputation
Materials failure /under specification
Lack of inspection
Lack of focus and lack of maintenance
Lack of production ITP and quality plan during construction
Standards need improving eg no Aust std
Under designed
Long span cantilever design
Treat the asset like a sign and not a bridge asset
Environment eg. Salt spray
Vehicle Impacts (Unlikely with controls in place)
Natural disaster events

Consequences

Loss of reputation

Key Controls	Control adequate? Select from below	Note(s)
Design		The unknown – "we don't know what we don't know" – add to treatment plan the investigation and recheck of the rating
Inspection L1, L2, L3?	Requires Improvement	Procedures adequate but budget is not adequate and full round has not been completed on first round – some difference of opinion – most conservative rating chosen
External Affairs process and ministerial and strategic & communication directorate	Adequate	
Verification of designs	Adequate	Variable L1/L2/L3 monitoring controls – depending on the CSS local environment will be needed
Construction testing and requirements	Adequate	Material testing quality has varied and past ITP during construction has been resolved
Maintenance and intervention in a timely manner from inspection reports	Inadequate	Funding and resources are the drivers
System of recording maintenance control include inventory records	Inadequate	Inventory does distinguish between gantries and cantilevers – use off line unsustainable processes and not updated yet.
New Control		Monitoring CSS performance as a unique group
Structural Asset Management plan(s) for major structures (on its way for CSS from the Alliance)		
Establish a budget to fix?		
Future control - Asset replacement planning , eg 30 years not 100 years structural design life and sign 50 years		Replacement cost can be lower than investigation and repair sometimes
Alliance Monitoring of baseline to understand infield performance, acceleration and fatigue stress/strain (Future waiting on the proposal)		$Regular overall CSS risk control effectiveness reviews will be needed

Effectiveness of controls

Adequate

Risk Review Date

21-Apr-21

FR3 - Loss of CSS integrity impacts transport services. Leads to congestion and community impacts.

Risk Owner: Major (4)

Return to Key Risks
Back to Heatmap
Treatment Plan

Risk Category

Service Delivery

Causes

Materials failure /under specification
Service Delivery
Lack of inspection
Lack of focus and lack of maintenance
Lack of production ITP and quality plan during construction
Standards need improving eg no Aust std
Under designed
Long span cantilever design
Treat the asset like a sign and not a bridge asset
Environment eg. Salt spray
Vehicle Impacts (Unlikely with controls in place)
Natural disaster events

Consequences

Loss of reputation
reduced amenity for traffic and road users

Key Controls	adequate? Select from below	Note(s)
Design		The unknown – "we don't know what we don't know" – add to treatment plan the investigation and recheck of the rating
Inspection L1, L2, L3?	Requires Improvement	Procedures adequate but budget is not adequate and full round has not been completed on first round – some difference of opinion – most conservative rating chosen
External Affairs process and ministerial and strategic & communication directorate	Adequate	
Verification of designs	Adequate	Variable L1/L2/L3 monitoring controls – depending on the CSS local environment will be needed
Construction testing and requirements	Adequate	Material testing quality has varied and past ITP during construction has been resolved
Maintenance and intervention in a timely manner from inspection reports	Inadequate	Funding and resources are the drivers
System of recording maintenance control include inventory records	Inadequate	Inventory does distinguish between gantries and cantilevers – use off line unsustainable processes and not updated yet.
New Control		Monitoring CSS performance as a unique group
Structural Asset Management plan(s) for major structures (on its way for CSS from the Alliance)		
Establish a budget to fix?		
Future control - Asset replacement planning , eg 30 years not 100 years structural design life and sign 50 years		Replacement cost can be lower than investigation and repair sometimes
Alliance Monitoring of baseline to understand infield performance, acceleration and fatigue stress/strain (Future waiting on the proposal)		§Regular overall CSS risk control effectiveness reviews will be needed

Effectiveness of controls

Adequate

Risk Review Date

21-Apr-21

FR4 - Loss of support for smart highways program, Mitigation controls in place, Challenging maintenance access

Risk Owner: Minor (2)

Return to Key Risks
Back to Heatmap
Treatment Plan

Risk Category

Service Delivery

Causes

Service Delivery
Materials failure /under specification
Lack of inspection
Lack of focus and lack of maintenance
Lack of production ITP and quality plan during construction
Standards need improving eg no Aust std
Under designed
Long span cantilever design
Treat the asset like a sign and not a bridge asset
Environment eg. Salt spray
Vehicle Impacts (Unlikely with controls in place)
Natural disaster events

Consequences

Loss of functionality of smart highway system and other cameras and sign from loss of CSS

Key Controls	adequate? Select from below	Note(s)
Design		The unknown – "we don't know what we don't know" – add to treatment plan the investigation and recheck of the rating
Inspection L1, L2, L3?	Requires Improvement	Procedures adequate but budget is not adequate and full round has not been completed on first round – some difference of opinion – most conservative rating chosen
External Affairs process and ministerial and strategic & communication directorate	Adequate	
Verification of designs	Adequate	Variable L1/L2/L3 monitoring controls – depending on the CSS local environment will be needed
Construction testing and requirements	Adequate	Material testing quality has varied and past ITP during construction has been resolved
Maintenance and intervention in a timely manner from inspection reports	Inadequate	Funding and resources are the drivers
System of recording maintenance control include inventory records	Inadequate	Inventory does distinguish between gantries and cantilevers – use off line unsustainable processes and not updated yet.
New Control		Monitoring CSS performance as a unique group
Structural Asset Management plan(s) for major structures (on its way for CSS from the Alliance)		
Establish a budget to fix?		
Future control - Asset replacement planning , eg 30 years not 100 years structural design life and sign 50 years		Replacement cost can be lower than investigation and repair sometimes
Alliance Monitoring of baseline to understand infield performance, acceleration and fatigue stress/strain (Future waiting on the proposal)		$Regular overall CSS risk control effectiveness reviews will be needed

Effectiveness of controls

Adequate

Risk Review Date

21-Apr-21

[Risk Number] - [CSS Failure] Treatment Action Plan Owner:	Treatment Action Plan Progress Tracking		
Treatment Action Plans prioritised for Implementation		**Task Owner:**	**Due Date:**
Structural Asset Management plan(s) for major structures (on its way for CSS from the Alliance). Linked with responsibility matrix.	N/A (Trial purposes only)	N/A (Trial purposes only)	N/A (Trial purposes only)
Establish a budget to fix & timelines for inspection types?	N/A (Trial purposes only)	N/A (Trial purposes only)	N/A (Trial purposes only)
Alliance Monitoring of baseline to understand infield performance, acceleration and fatigue stress/strain (Future waiting on the proposal)	N/A (Trial purposes only)	N/A (Trial purposes only)	N/A (Trial purposes only)
Setup frequencies of control effectiveness checks	N/A (Trial purposes only)	N/A (Trial purposes only)	N/A (Trial purposes only)
Audit program to ensure treatment program is adhered to	N/A (Trial purposes only)	N/A (Trial purposes only)	N/A (Trial purposes only)
Consider some of these treatements if applicable later	N/A (Trial purposes only)	N/A (Trial purposes only)	N/A (Trial purposes only)
Consider vibration monitoring or cyclical monitoring - sensible.	N/A (Trial purposes only)	N/A (Trial purposes only)	N/A (Trial purposes only)
Damping devices could be considered on structures and when structures have concerns with winds	N/A (Trial purposes only)	N/A (Trial purposes only)	N/A (Trial purposes only)
Consider standards improvement ? (Not likely as it is an external issue)	N/A (Trial purposes only)	N/A (Trial purposes only)	N/A (Trial purposes only)

Return to Key Risks
Go to cause/consequence themes

Appendix C WA Road Research & Innovation Program (WARRIP) Project Concept

Version	**1.0**
ARRB Ref. No.	
Prepared by	

Working Title	**WARRIP Project Concept Costing Maintenance Monitoring of CSS Structures**
Project Champion / Project Source	*The project champion at MRWA is Mahes Rajakaruna who requires this project concept as a follow-on from Project 2020-0016.*
What is the problem/issue that the project seeks to address?	*Previously, the Project 2020-0016 proposed a theoretical risk-based prioritisation system for assigning, monitoring and maintenance activities for Cantilever Sign Support (CSS) structures. Currently, there is an incomplete understanding of the risk posed by CSS structure assets within the asset group to practically implement findings from the previous project. Fatigue performance of the structures is not measured currently and caused uncertainty during the risk workshop conducted in April 2021 for Project 2020-0016.*
Objective of the Project	*The overall objective of the project is to establish effective risk monitoring to support the integrity of CSS maintenance and asset management. The project will establish baseline performance metrics for a CSS structure. This will support the categorisation of structures ready for a risk assessment and provide inspection trials to support MRWA structural integrity assurance and maintenance planning. This project will help reduce the uncertainty in asset decision making processes and underpin MRWA's due diligence in relation to these assets.*
Output/s of the Project	*The project will provide monitoring to improve the understanding of the CSS structural assets across Main Roads Perth Road Networks and reduce the uncertainty for making maintenance and replacement budget decisions.* *The Phase 1 output of the project will be:* • *A complete list of Main Roads CSS structures.* • *The baseline list of CSS assets prioritised using risk reviews ready for Main Roads to earmark assets either for replacement or for maintenance activities.* • *Costing for the behavioural 'fingerprint' performance test of one CSS structure using appropriate sensors – ideally accelerometers and strains gauges.* • *DVR[2] and Level 1 inspections provided for the trial.* *The Phase 2 output of the project: 3-5 years performance data ready to compare to CSS sign design criteria.*
Benefits to Main Roads	*Through implementing the findings of this project, Main Roads will have a clearer understanding of the asset base risk and be able to understand and make better maintenance and disposal replacement prioritisation decisions. In addition, through having a greater understanding of the asset base list, Main Roads will be able to plan and budget with more certainty.*

[2] DVR: Digital Video Road – a tool that allows high resolution Hawkeye video data, collected from road inspection vehicles, to be viewed on a personal computer.

Methodology	*Methodology for the project is as follows:* ***Phase 1*** • *Complete list of Main Roads CSS structures, including a DVR video assessment of assets against database and face to face consultation with various Main Roads personnel to discuss CSS assets.* • *The baseline list of CSS assets prioritised using risk reviews ready for Main Roads to earmark assets either for replacement or for maintenance activities. This will involve a number of risk sessions with stakeholders to categorise and gain consensus on categorisation with engineering judgement supplied by key Main Roads personnel.* • *Scoping and costing of the behavioural 'fingerprint' performance test of one CSS structure using appropriate sensors – ideally accelerometers and strains gauges – will be investigated, including benchmarking of similar processes used elsewhere.* • *A team will review Main Roads DVR road inspection videos to establish a process for DVR assessment. From this, the prioritisation for Level 1 trial and indication of asset condition will be tabled to Main Roads at the conclusion of year 1 phase.* • *A trial Level 1 inspection provided for the trial will be conducted with trial review of results approved by Main Roads.* • *Once the inspection trials have been completed, the Main Roads sign Inspection guidelines will be updated based on learnings from the trials.* • *Develop cost benefit analysis for the future physical monitoring program of CSS.* • *Finally, in Phase 1, Reporting, Benefit/Cost Analysis, Presentation/Dissemination Activities, Project Summary will be developed and reported back to Main Road.* *The **Phase 2** output of the project: 3-5 years performance data ready to compare to with CSS sign design criteria. Annual reports will be developed characterising the accelerometer and strain gauge readings ideally correlated with sign wind loadings from an environmental perspective. E.g., Wind velocity data.*
Strategic Links / **Key Stakeholders**	*Austroads project number ABT6196 was completed in January 2021 (Austroads 2021). This project was design focussed and did not cover asset monitoring or life of asset integrity. The project did not deliver maintenance strategy or risk-based prioritisation of maintenance activities. Furthermore, the ABT6196 delivery presentation by Austroads in June 2021 called out the need for both strategy and maintenance planning.*
Related Work	*This project is a follow-on project providing support for the findings from the WARRIP project 2020-016 (ARRB ref. Number 015863). The prior work established a risk-based methodology to manage the CSS sign structures. The prior project provided an updated knowledge base for Main Roads to manage existing cantilever structures, influence the design and construction of future similar structures and improve the understanding of risk controls. The prior work recommended varying maintenance inspections and cost using a risk-based prioritisation process along with regular control effectiveness tests.*
Estimated Cost* *(Excl. GST)*	***Phase 1 (Year 1)*** *Investigate and establish current CSS asset base using video, update CSS asset register $20,000* *Implement risk-based framework and categorisation of CSS asset base into integrated risk management systems at Main Roads $25,000* *Inspection costs* (Each)* • *Assessment of DVR inspection videos $50 each CSS (Additional if new videos are required).* • *Level 1 $200 each.* • *Level 2 $1000 each (with no access restrictions or equipment required).* • *Level 3 $P.O.A. (likely this will not be performed assuming most Level 3s will be deferred to earlier CSS replacement strategy).*

	Update the Main Roads Inspection guidelines $50,000 *Establish fingerprint benefit/cost ratio costing $10,000 for accelerometer/strain gauge trial Phase 2.* ***Phase 2 (Year 2 onward)*** *Establish accelerometer/strain gauges test for CSS structure with no redundant load paths and asymmetrical loading and test over a 3-5-year period:* • *Setup $100,000* • *Annual monitoring cost $50,000 per year.* ****Excludes travel from the eastern seaboard and reimbursables. Assumes resourcing sourced from Perth.***
Estimated Duration *(Not including project development and approval processes)*	*Phase 1: 12 months* *Phase 2: 3-5 years completed*