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Standardised Bridge Barrier Design Guidelines



Guideline AP-G108-25



Standardised Bridge Barrier Design Guidelines

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Abstract

These guidelines provide direction on the standardised design of bridge traffic barriers across Australia and New Zealand. They address inconsistencies in current practice and update the previously available *Standardised Bridge Barrier Design* (Austroads 2013) by incorporating current research, guidance, and standards.

Inconsistencies in the specification and design of traffic barriers for bridges have been identified as a major issue in current practice throughout Australia and New Zealand. The need for standardised solutions has been recognised by road authorities, consultants and contractors. The main areas of concern include determining appropriate barrier performance levels, structural design criteria, the absence of standard barrier design details, retrofitting guidance for existing bridge barriers, bridge transition approach barriers, barrier end treatments and protection for overpass bridge support.

By providing a unified approach, these guidelines aim to support safer, more consistent and cost-effective bridge barrier designs across jurisdictions.

Keywords

Bridge barrier, barrier design, median barrier, approach barrier, support protection barrier, retrofit of barrier

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About Austroads

Austroads is the association of Australasian transport agencies.

Austroads' purpose is to support our member organisations to deliver an improved Australasian road transport network. To succeed in this task, we undertake leading-edge road and transport research which underpins our input to policy development and published guidance on the design, construction and management of the road network and its associated infrastructure.

Austroads provides a collective approach that delivers value for money, encourages shared knowledge and drives consistency for road users.

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- Transport for NSW
- Department of Transport and Planning (Transport Victoria)
- Queensland Department of Transport and Main Roads
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- Department for Infrastructure and Transport South Australia
- Department of State Growth Tasmania
- Department of Logistics and Infrastructure Northern Territory
- Transport Canberra and City Services Directorate, Australian Capital Territory
- Department of Infrastructure, Transport, Regional Development, Communications, Sport and the Arts
- Australian Local Government Association
- NZ Transport Agency Waka Kotahi.

Summary

Inconsistencies in the design of traffic barriers for bridges have been identified as a major issue in current practice throughout Australia and New Zealand. The need for standardised solutions was recognised by road authorities, consultants and contractors. The main areas of concern are the determination of appropriate barrier performance levels, structural design criteria, the lack of standard barrier design details, guidance on retrofitting existing bridge barriers, bridge approach barriers and overpass bridge support protection. This report updates the Austroads research report *Standardised Bridge Barrier Design* (Austroads 2013), based on current research, guidance and standards.

This report includes the following guidelines and key features:

- bridge barrier design procedures and considerations
- selection of bridge barrier performance levels, introducing barrier specification standards to facilitate the selection of barrier performance levels based on the provisions of AS 5100.1:2017
- types of bridge barriers and considerations when choosing the material used
- identification of the length of need required to protect vehicles from roadside hazards
- discussion of the transition and end treatments required for bridge barriers
- the design criteria for bridge barriers
- considerations for barriers in the median of bridges
- retrofitting of existing bridge barriers, providing a methodology for the selection of appropriate barrier performance levels and evaluation of existing bridge structures
- discussion on alignment of bridge barriers with the Safe System and how to assess
- bridge support protection barriers, providing a methodology for determining the level of protection, selection of barrier performance level and barrier layout design including examples
- process and considerations for modelling of bridge barrier performance.

Acronyms

Acronym	Description
AADT	Annual Average Daily Traffic
AASHTO	American Association of State Highway and Transportation Officials
AGRD	Austroads Guide to Road Design
AS	Australian Standard
ASBAP	Austroads Safety Barrier Assessment Panel
BCR	Benefit Cost Ratio
BTTF	Austroads Bridge Technology Task Force
FE	Finite Element
FEA	Finite Element Analysis
FEM	Finite Element Modelling
FHWA	The Federal Highway Administration
LON	Length of Need
MASH	Manual for Assessing Safety Hardware
ROW	Right of Way
SSA	Safe System Approach
TL	Test Level
WRSB	Wire Rope Safety Barriers

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1. Introduction

1.1 Purpose

The purpose of this document is to consolidate current guidance for bridge barrier design and update Austroads (2013) based on latest standards, practice and literature. The document outlines the bridge barrier design process and provides examples for key steps. When using this guideline, local publications should also be checked for additional requirements.

1.2 Scope

This report updates the guidance provided in Austroads (2013) based on national and international guidance for barriers related to bridges. This includes road bridge barriers, bridge approach barriers including transition barriers and end terminals, barriers for pier protection for road over road and road over rail overpasses. Noise barriers, pedestrian barriers, bicycle path barriers or barriers on railway or pedestrian bridges are out of the scope of this report.

1.3 Safe System considerations

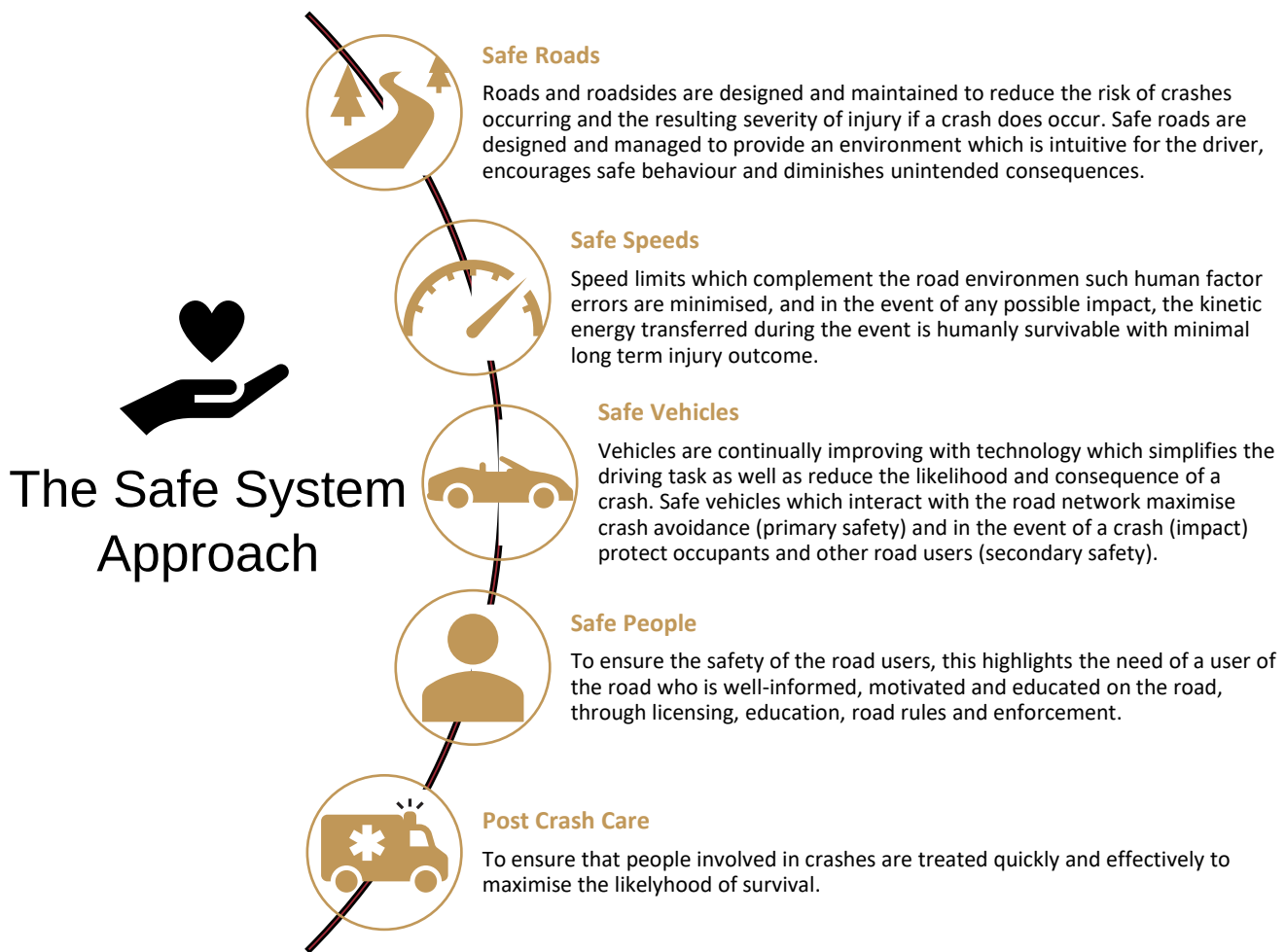
1.3.1 Safe System approach

The Safe System approach is a road safety approach which acknowledges that errors will occur on the road, as humans are fallible, which can cause unintentional death and injury. There are 4 key principles which form the basis of the Safe System philosophy (Austroads 2021a):

1. People make mistakes that can lead to road crashes.
2. The human body has limited physical ability to tolerate crash forces before harm occurs.
3. A shared responsibility exists amongst those who plan, design, build, manage and use roads and vehicles and those who provide post-crash care to prevent crashes resulting in serious injury or death.
4. All parts of the system must be strengthened to multiply their effects, so that if one-part falls, road users are still protected.

The underlying cornerstones of the approach are illustrated in Figure 1.1.

Figure 1.1: Cornerstones of the Safe System approach



Source: Based on Austroads (2021a).

1.3.2 Safe System considerations for the design of bridge barriers

The design of the bridge barriers is integral to the function of the Safe System approach, particularly the Safe Road cornerstone.

The barrier placed on a bridge, on approach to a bridge or adjacent to a bridge support has 2 fundamental safety functions:

1. To protect nearby road users from the significant hazard posed by a vehicle falling from a bridge or overpass. In terms of crash severity, a falling vehicle represents a considerable amount of energy. Injuries sustained by another road user impacted by a falling vehicle are foreseeably catastrophic. The breach of a vehicle over or through a bridge barrier places considerable stress on the post-crash response element. Emergency responders arriving at a crash scene where a vehicle has gone off a bridge are faced with considerable health and safety risk as well as logistical difficulties involved in rescuing injured people and righting the vehicle.
2. To protect the occupants within the errant vehicle from sustaining injury. The barrier must be designed to minimise the risk that the driver or other occupants travelling within an errant vehicle will be seriously injured. Thus, the approach barrier and bridge barrier combination should be designed to contain (and safely redirect) an errant vehicle onto the roadway without significant risk of injury. The barrier should be designed to minimise the risk that an errant vehicle will travel around the barrier and experience a significant drop. The bridge supports should be designed to minimise the risk that the bridge support will

be impacted. Therefore, the barrier adjacent to the support must consider applicable barrier impact deflection requirements.

The design of bridge barriers poses a significant challenge. The barrier must be rigid to prevent the likelihood that a vehicle impacting the structure will breach the structure and fall over the side of the bridge. Adding too much stiffness places the bridge structure (particularly at the connection) under significant stress and is likely to result in unrecoverable structural damage to the entire bridge, as bridge barriers require elements to remain connected and in place in the event of an impact. This is a particular problem for retrofit designs, where the structural capacity of the bridge deck may be unknown.

For an occupant impacting a barrier and for the bridge deck, a flexible design that absorbs crash energy is preferable. Unfortunately, many bridges do not have the room to allow for flexible designs to be installed with sufficient deflection zone clearance provided. Trials of flexible, performance-based designs on bridges that demonstrate greater alignment with Safe System approach objectives (such as the Dolre TL-5 bridge barrier, see Figure 1.2 for example) are currently ongoing.

Figure 1.2: Simulation of Dolre TL5 design according to the Manual for Assessing Safety Hardware (MASH)



Source: Evaraert (2019).

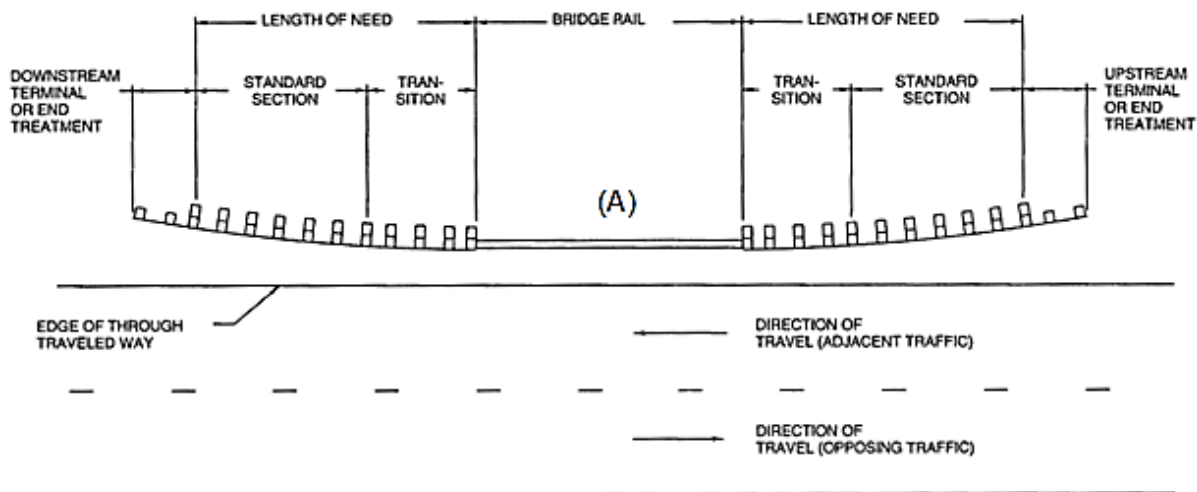
2. Bridge Barrier Design Process

This guide outlines the steps and processes involved in designing barriers for bridges. The guide has been broken up into the aspects that need to be considered in the design process, which include:

- Required barrier performance (Section 3) – Steps involved in identifying the barrier performance level required for the bridge, based on the traffic volume and composition.
- Barrier type (Section 4) – Identification of the different types of bridge barriers commonly used.
- Barrier design (Section 5) – Design criteria for the design of the barriers.
- Bridge approach barriers (Section 6) – Identification of transition and end treatments required for bridge barriers.
- Bridge support protection barrier (Section 7) – Criteria for the protection of bridge supports.
- Retrofit of barrier (Section 8) – Process and considerations for replacing barriers on existing structures.
- Risk based approach (benefit cost) (Section 9) – Taking a risk-based approach to assess the costs and benefits associated with installing various barrier designs.
- Manual for Assessing Safety Hardware (MASH) Full-scale crash testing (Section 10) – Considerations on conducting a full-scale crash test in accordance with MASH criteria and results of testing a TfNSW medium barrier.
- Finite element analysis (Section 0) – Process and considerations for modelling of bridge barrier performance.

Figure 2.1 shows a general layout of a bridge barrier, bridge approach barrier, transition and end treatment.

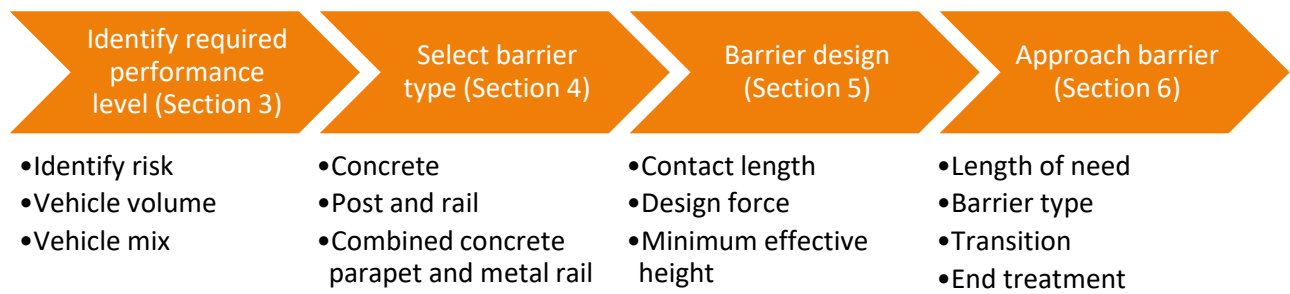
Figure 2.1: Bridge barrier component layout



Source: AASHTO (2011).

The process and considerations for the design of a barrier (including bridge, median and support barriers) are provided in Figure 2.2.

Figure 2.2: Barrier design process



3. Identify Required Performance Level

The general procedure involves identifying a site condition category, which then determines the appropriate methodology for selecting a site-specific barrier performance level.

3.1 Bridge barrier performance levels

The Australian Standard AS 5100.1:2017 specifies the performance levels as: No barrier, Low, Regular, Medium, and Special, and identifies their equivalent MASH (AASHTO 2016) test levels. In Table 3.1, the general containment requirements and the equivalent MASH test levels (AASHTO 2016) are presented.

Table 3.1: Bridge barrier performance levels

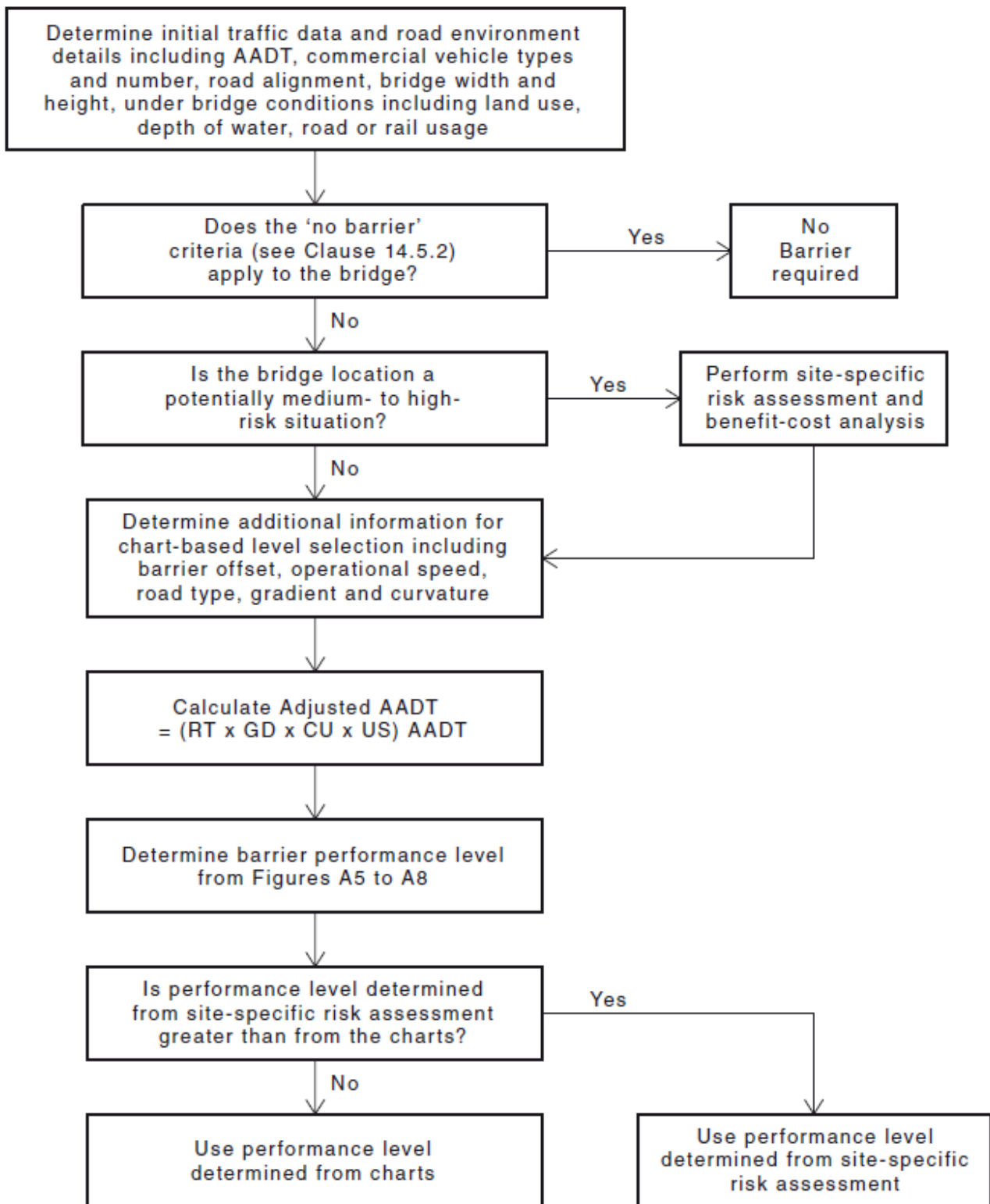
Barrier performance level	MASH test level	Criteria for site condition	Test vehicle	Test speed (km/h)	Impact angle (dg)
No barrier	N/A	Traffic barriers may be omitted provided all the following conditions apply: • Bridge deck is less than 1.5 m above the ground or invert level of the waterway. • Traffic volumes are less than 150 vehicles per day. • Radius of curvature of bridge is such that road approaches have sight distance greater than the stopping distance. • The width between kerbs is not less than 6.5 m for a two-lane bridge or 4.2 m for a single-lane bridge. • The edge of the bridge is at least 1.0 m from the edge of traffic lanes. • No pedestrian traffic is anticipated. • Any water beneath the bridge is normally less than 1.2 m deep. • The provision of barriers would prevent the passage of debris or the barriers would be frequently damaged by heavy debris or both.	N/A	N/A	N/A
Low	TL2	• Bridges on roads with low traffic volumes. • Bridges with low to medium height above ground or water. • Bridges with an essentially straight alignment. • Bridges with a width between barriers of not less than 6.5 m for a two-lane bridge or 4.2 m for a single lane bridge.	Car (1.1 t)	70	25
			Utility (2.27 t)	70	25
Regular	TL4	Regular performance barriers shall be provided for effective containment of general traffic on all roads unless a lower or higher performance level is required.	Car (1.1 t)	100	25
			Utility (2.27 t)	100	25
			Single unit rigid truck (10 t)	90	15

Barrier performance level	MASH test level	Criteria for site condition	Test vehicle	Test speed (km/h)	Impact angle (dg)
Medium	TL5	For roads with medium- to high-volume of heavy vehicles, site-specific justification shall be based on a risk assessment, with emphasis on third party risk including, but not limited to: - bridges over major roadways - bridges over high frequency passenger rail lines or goods lines carrying noxious, flammable or large volumes of freight, or over critical rail infrastructure - bridges over high occupancy land use. For roads with medium- to high-volumes of buses, site-specific justification shall be based on a risk assessment, with emphasis on the occupants of buses for situations including, but not limited to: - bridges more than 10 m high - bridges over water normally more than 3 m deep - bridges over major roadways - bridges over high frequency passenger rail lines or goods lines carrying noxious, flammable or large volumes of freight, or over critical rail infrastructure - bridges over high occupancy land use.	Car (1.1 t)	100	25
			Utility (2.27 t)	100	25
			Articulated van (36 t)	80 (MASH)	15
				90 (AS 5100)	
Special	TL6+	Shall be provided where specified by relevant authority. These shall be provided for: - effective containment of heavy, high centre of gravity vehicles in high-risk situations on high-speed freeways, major highways and urban arterial roads with a high volume of mixed heavy vehicles - site-specific, unusual conditions at critical locations - locations where it is essential that penetration or vaulting by vehicles specified by the relevant authority under impact conditions need to be prevented. Site-specific justification shall be based on risk assessment, with emphasis on third party risk for situations including, but not limited to, high risk, critical adjacent facilities such as: - power stations - gas tanks - oil refineries - communication facilities.	Car (1.1 t)	100	25
			Utility (2.27 t)	100	25
			Articulated tanker (36 t) or articulated van (44 t)	80 (MASH)	15
				100 (AS 5100)	

Source: Based on AASHTO (2016) and AS 5100.1:2017.

The procedure used for selecting the barrier performance level between Low, Regular, and Medium using site-specific traffic data and road environment details, based on Appendix A of AS 5100.1:2017, is presented in Figure 3.1. The steps are detailed in the following sub-sections.

Figure 3.1: Barrier performance level selection



Source: AS 5100.1:2017.

3.2 Bridge support protection barrier performance level

The selection of the bridge support protection barrier performance level is discussed further in Section 7.3. Overall, the process is similar to the selection of the performance level for bridges.

3.3 Determine adjusted AADT

The adjusted AADT, as noted in Figure 3.1, is calculated using Equation 1.

$$\text{Adjusted AADT} = (\text{RT} \times \text{GD} \times \text{CU} \times \text{US}) \text{AADT} \quad 1$$

where

RT = road type adjustment factor, derived from Table 3.2 (Table A1, AS 5100.1:2017)

GD = road grade adjustment factor, derived from Figure 3.2 (Figure A2, AS 5100.1:2017)

CU = road curvature adjustment factor, derived from Figure 3.3 (Figure A3, AS 5100.1:2017)

US = deck height and under-structure condition adjustment factor, derived from Figure 3.4 (Figure A4, AS 5100.1:2017)

Table 3.2: Road type factor (RT)

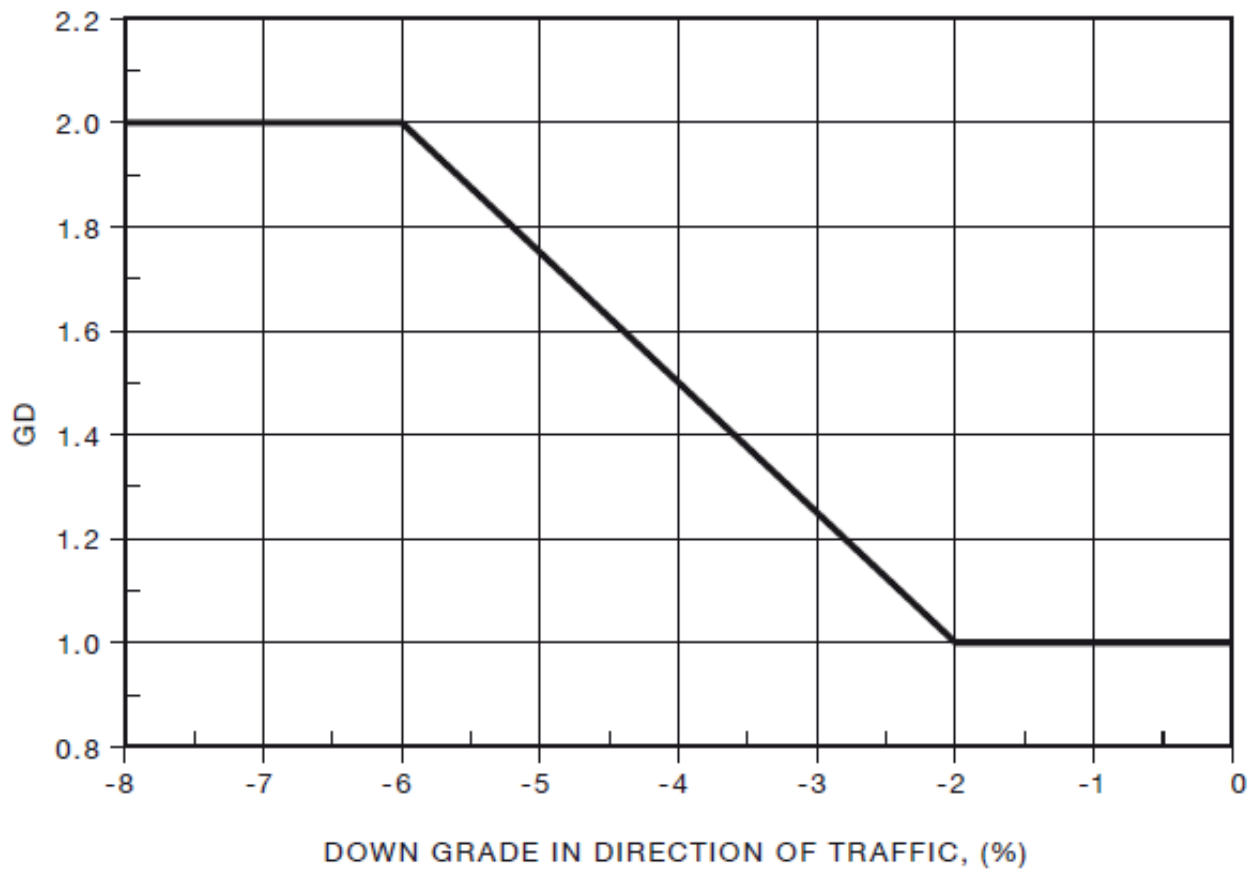
Road type	AADT	Number of lanes*	Adjustment factor RT
One-way**	Based on one-way traffic	1 or more	2.0
Two-way divided	Based on two-way traffic	2 or more	1.0
Two-way undivided	Based on two-way traffic	2 or more	1.5

*The number of lanes is the total number of lanes on the bridge.

**Includes one-way roads, freeway ramps and bridges on separated carriageways of freeways and highways.

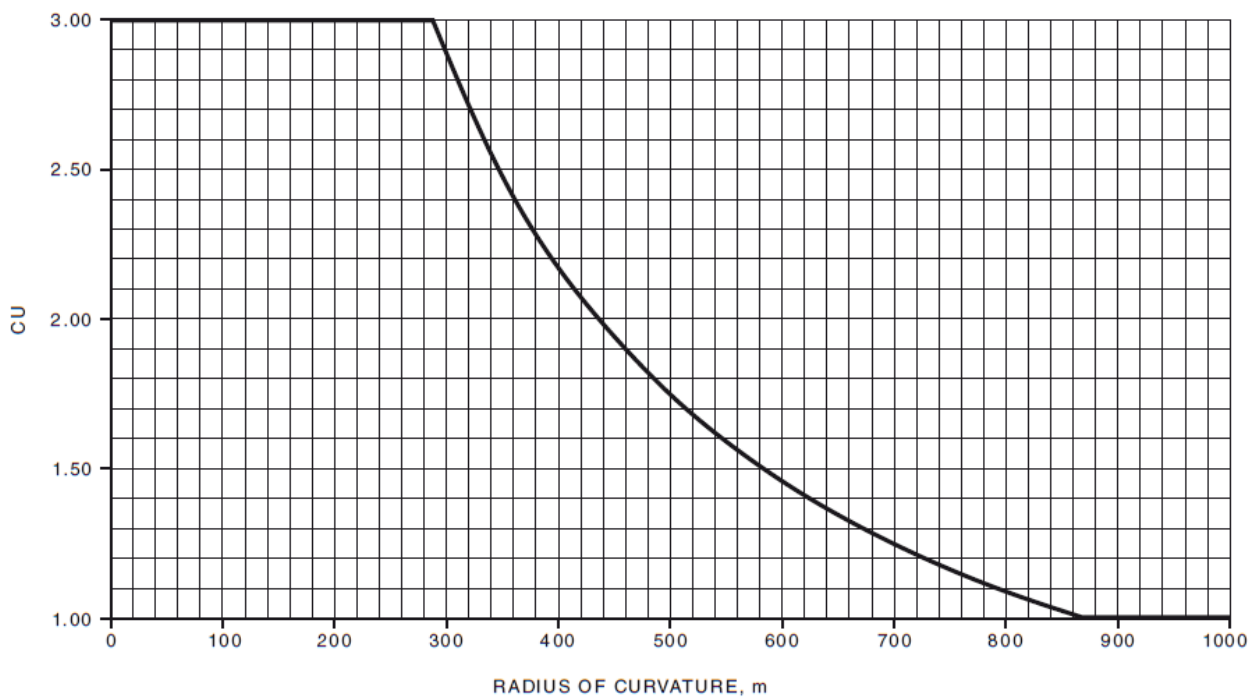
Source: AS 5100.1:2017.

Figure 3.2: Grade factor (GD)



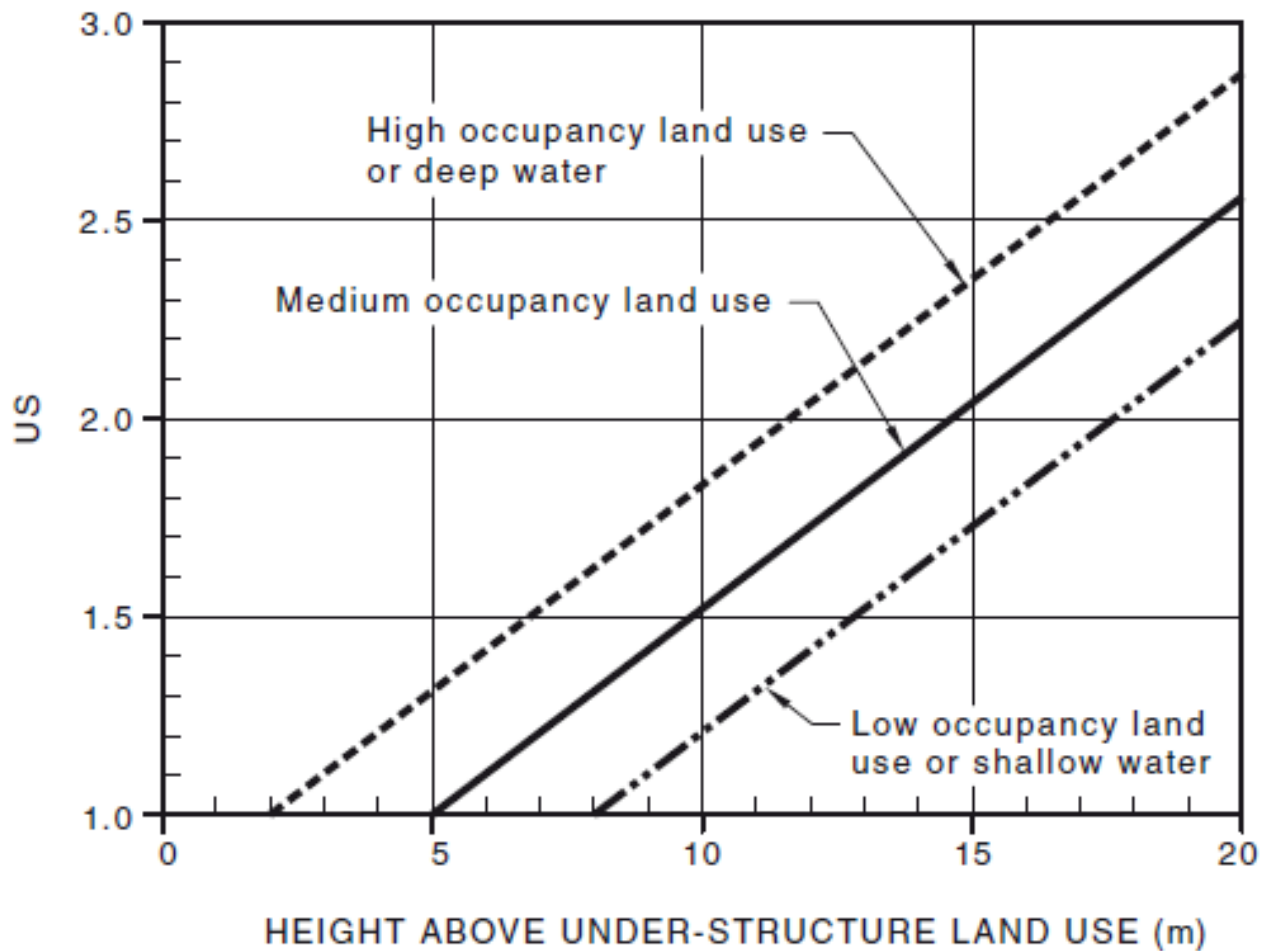
Source: AS 5100.1:2017.

Figure 3.3: Curvature factor (CU)



Source: AS 5100.1:2017.

Figure 3.4: Deck height and under-structure (US) conditions

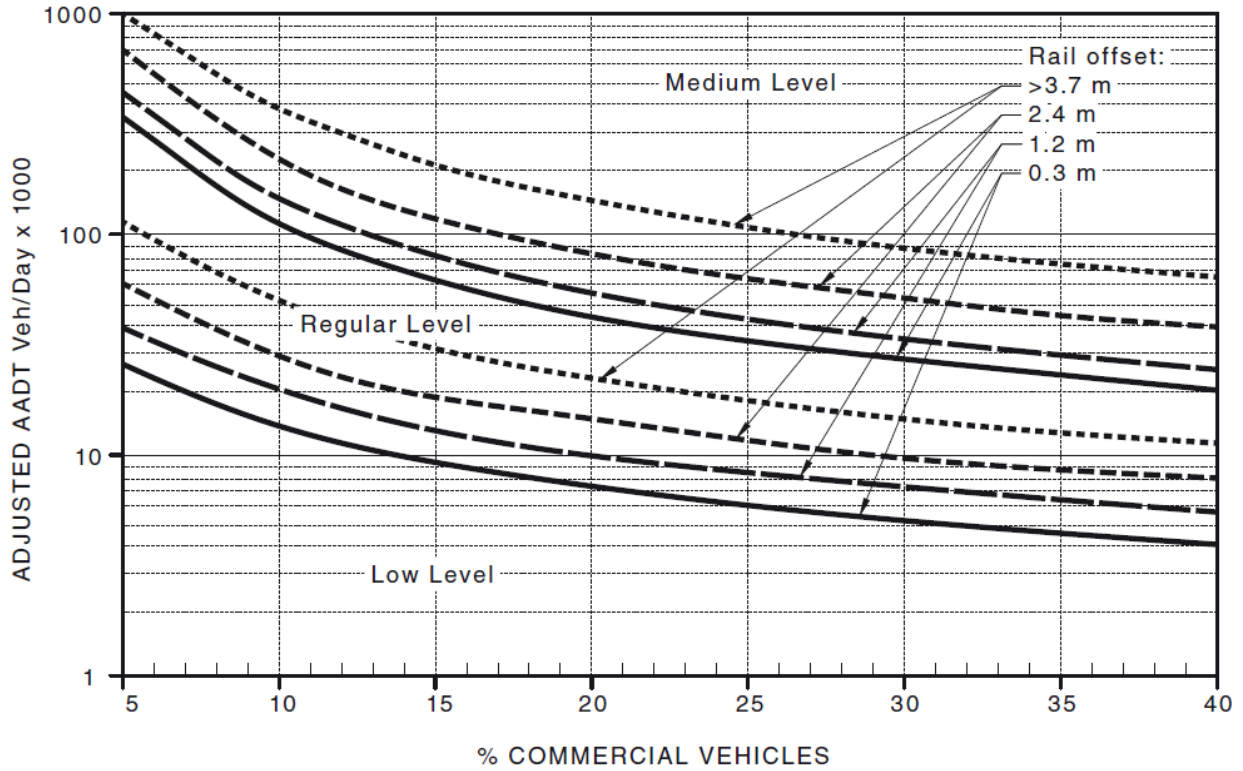


Source: AS 5100.1:2017.

3.4 Determine barrier performance level

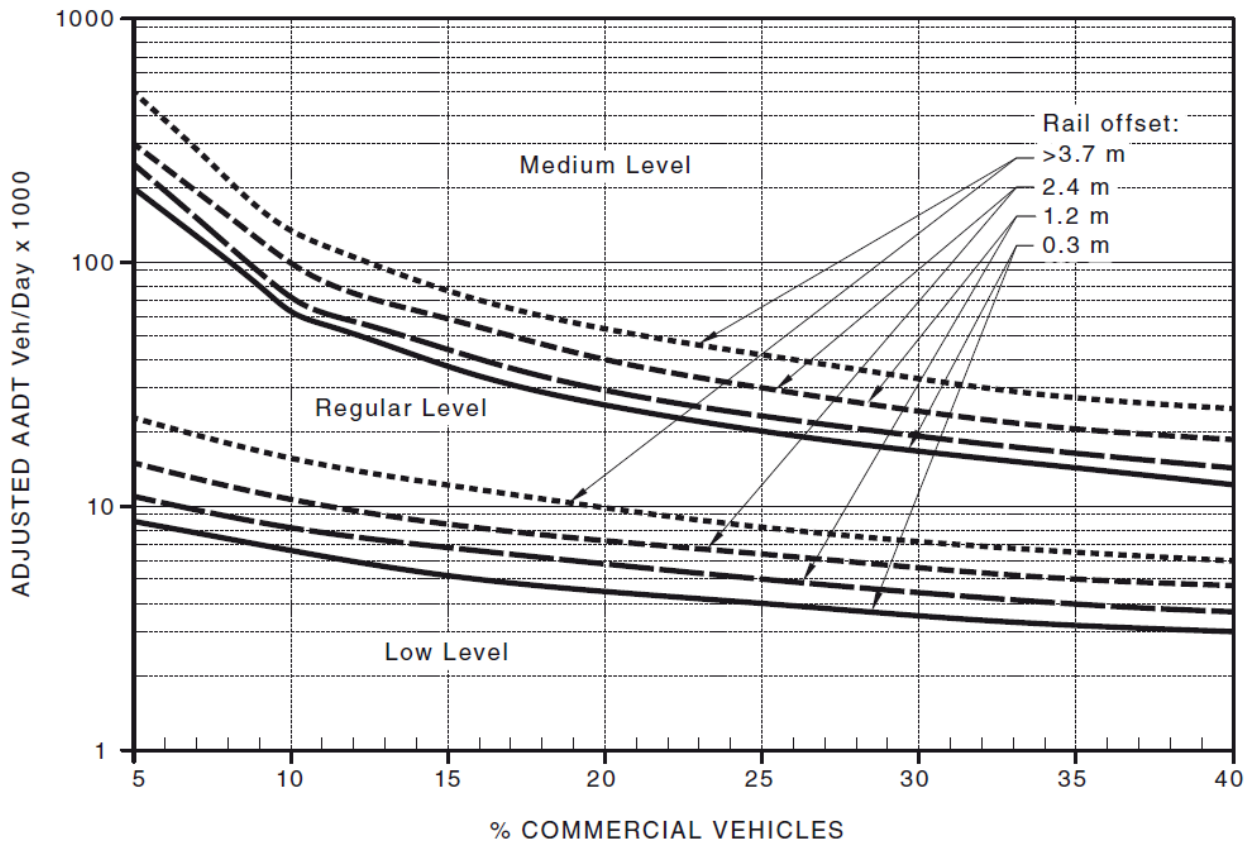
The barrier performance level will be determined from Figure 3.5 to Figure 3.8, based on the design speed, adjusted AADT (refer to Section 3.3), proportion of commercial vehicles and rail offset.

Figure 3.5: Threshold limits, 60 km/h



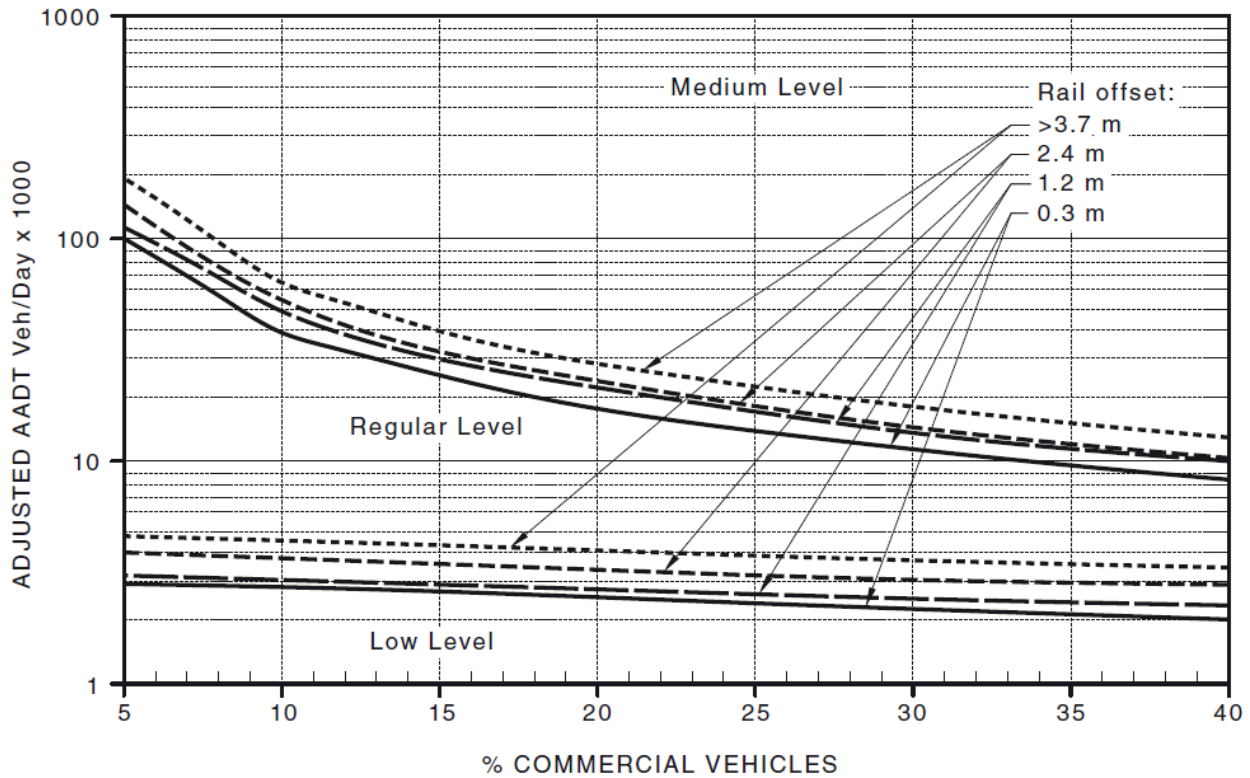
Source: AS 5100.1:2017.

Figure 3.6: Threshold limits, 80 km/h



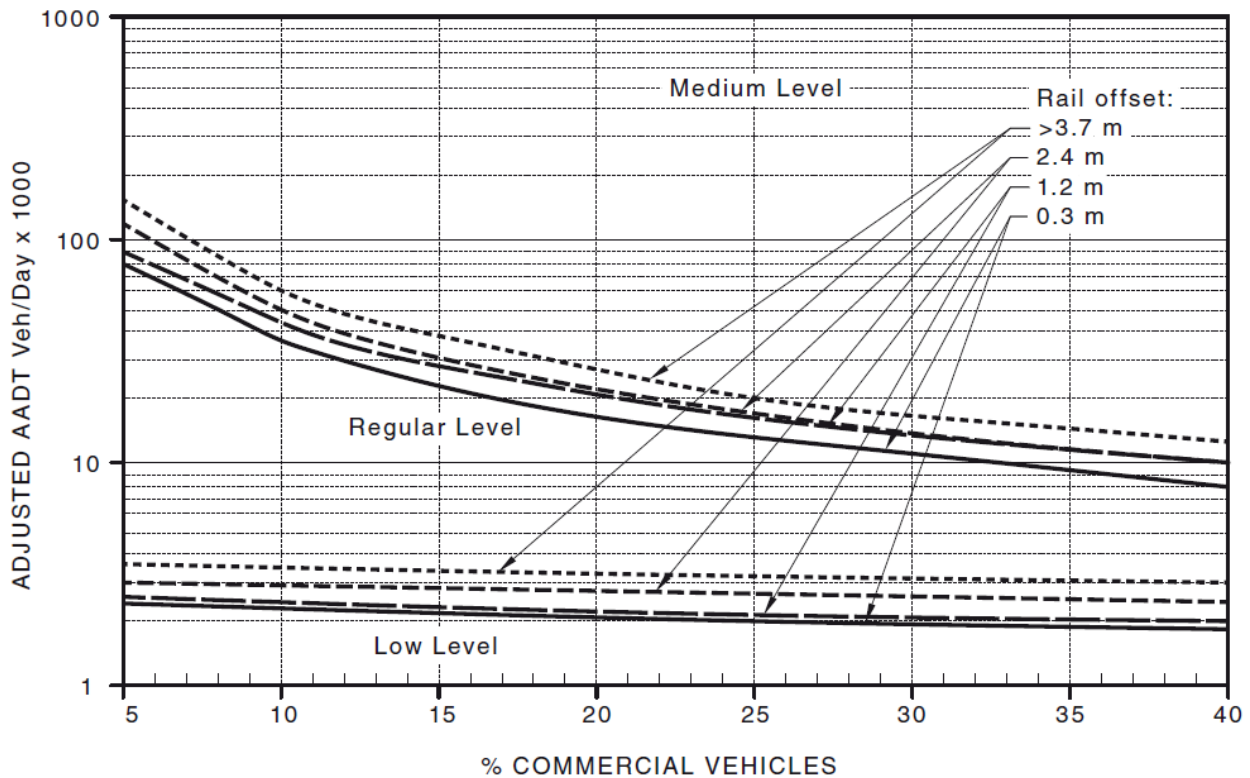
Source: AS 5100.1:2017.

Figure 3.7: Threshold limits, 100 km/h



Source: AS 5100.1:2017.

Figure 3.8: Threshold limits, 110 km/h



Source: AS 5100.1:2017.

4. Barrier Type

There are numerous bridge barrier types and designs for each performance level of bridge barrier. The barrier type is generally chosen based on the performance and the available materials. The 3 main types of bridge barriers include:

- concrete barriers (Section 4.1)
- post-and-rail barriers (Section 4.2)
- combined concrete parapet and metal rails (Section 4.3).

This section also outlines the acceptance criteria for non-standard bridge traffic barriers, including proprietary barriers (refer to Section 4.4).

4.1 Concrete barriers

Typical cross-sections of bridge concrete barrier include:

- vertical wall
- single slope
- F-shape and New Jersey.

The properties of these types of barriers are discussed below.

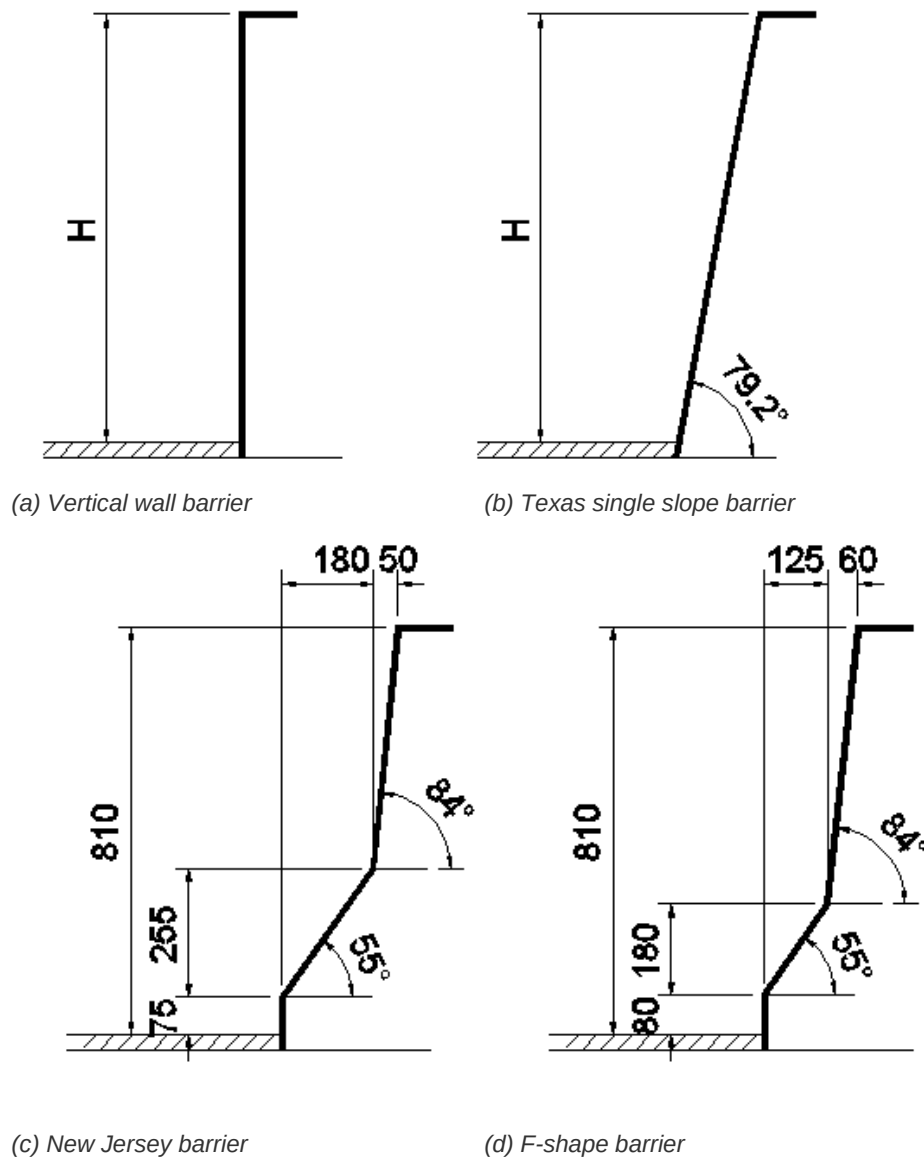
Based on AS 5100.1, an F-shape or single slope barrier shall be used, unless approved otherwise by the relevant authority.

4.1.1 Vertical wall barrier

Vertical wall concrete barriers (Figure 4.1a) do not lift the vehicle and hence do not have the energy management feature of the F-shape, single slope or New Jersey barriers. Vertical wall concrete barriers can be an effective alternative to the wider safety-shape barriers and can preserve available median width at narrow locations such as in front of bridge piers. Vehicle damage in crashes with a vertical wall is greater than with safety-shaped barriers, but injuries are comparable, and preservation of shoulder width is a safety benefit (AASHTO 2011). In a crash with a vertical wall all 4 wheels typically remain on the ground, and this minimises the potential for vehicles to roll over. The trajectory of cars after they crash into vertical walls is also uncertain because wheel damage may occur as the axle makes contact with the barrier. Additionally, due to the shape of the barrier there is the possibility of the head of the vehicle occupants seated closest to the wall striking the barrier.

For these reasons vertical wall barriers are not preferred. However, they may be suitable in some situations on urban roads where road width is highly constrained or on low-speed roads where the appearance of the road environment is important; for example, where it is desired to construct or face the wall using natural stone or some other material. An advantage of the vertical wall is that its profile is not affected by re-sheeting or resurfacing of the pavement.

Figure 4.1: Typical types of concrete barriers



Note: New Jersey barriers are no longer installed in Australia, but some remain in service.

4.1.2 Single slope barrier

Single slope barriers (Figure 4.1b) have a more consistent performance than vertical wall barriers. Single slope barriers can also facilitate pavement resurfacing without the profile being adversely affected.

Two types of single slope barriers have been developed. The Texas barrier has a wall slope of 10.8° and the Californian has a wall slope of 9.1° . These were developed by the Texas Transportation Institute and the California Department of Transportation respectively.

Crash tests have indicated that the performance of the Texas single slope barrier is comparable to the New Jersey barrier and that the performance of the Californian single slope barrier is comparable to that of the F-shape barrier. In approving the use of single slope barriers, the US Federal Highway Administration (FHWA) suggested that the Californian single slope barrier is an improvement over both the standard New Jersey concrete barrier shape and the Texas single slope barrier because of the reduced vehicular climb seen upon impact and the less severe post-crash vehicular trajectories observed in crash test videos.

4.1.3 F-shape barrier

The F-shape barrier (Figure 4.1d) has a similar profile to the New Jersey barrier, the main difference being that the height of the lower sloped surface is smaller. The major performance difference is that the lower slope of the F-shape profile significantly reduces the lifting of an impacting vehicle, resulting in a reduced tendency for vehicles to roll, particularly small cars.

4.1.4 New Jersey barrier

The New Jersey barrier (Figure 4.1c) has been used throughout Australia and has provided satisfactory service. For common shallow-angle impacts the shape is intended to minimise sheet metal body damage by allowing the vehicle tyres to ride up on the lower sloped face. Energy is dissipated by lifting and lowering of the vehicle, compression of the vehicle suspension and deformation of the body of the vehicle. While New Jersey barriers remain in service, the shape was modified to the F-shape barrier to address the likelihood of vehicle rollover when impacted by small to medium vehicles. Consequently, jurisdictions generally no longer install New Jersey barriers.

For higher impact angles the New Jersey shape results in a staged response by an impacting vehicle, namely:

- The vehicle bumper impacts the upper sloped face and slides upwards, lifting the vehicle.
- As the vehicle becomes more parallel with the barrier, the wheel contacts the lower sloped face causing additional lift through compression of the front suspension.
- The lifting reduces friction between the tyres and the paved surface, and this facilitates banking and redirection of the vehicle.

Excessive lifting of the vehicle may cause it to yaw, pitch or roll during contact with the barrier, and to roll over when the tyres make contact with the road again. As wheel side-rubbing forces can provide additional lift, exposed aggregate and other rough surfaces should be avoided.

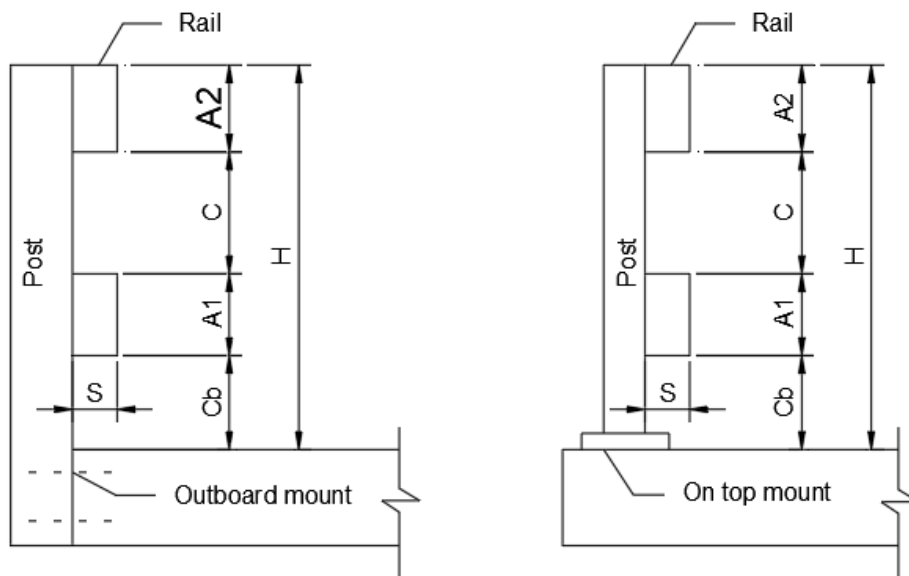
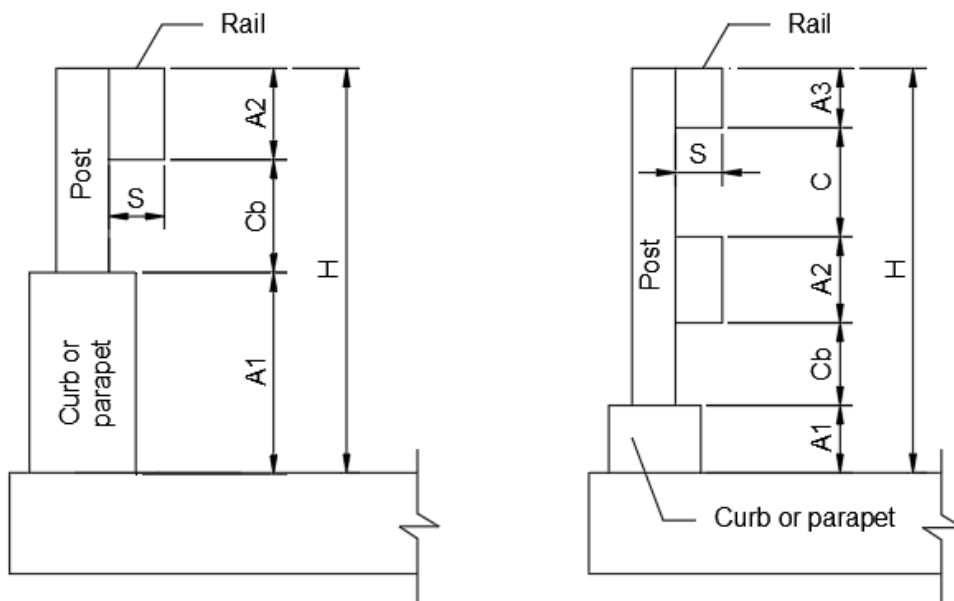
The 75 mm high vertical face at the base of the New Jersey barrier is intended to provide an allowance for future pavement overlays. Apart from increasing the extent to which a vehicle is lifted, this vertical face plays no significant role in the performance of the barrier.

4.2 Post-and-rail barriers

Post-and-rail barriers have been used mainly for Low and Regular performance levels (Figure 4.2). The components of this barrier type include:

- Posts: Steel hollow box or channel sections, connected to the bridge deck by bolts.
- Rails: Steel hollow box or W-beam or thrie-beam sections bolted to the posts. There may be one, two- or three-rail configurations.

Figure 4.2: Post-and-rail barriers

*Metal or timber rail**Concrete and metal rail*

4.3 Combined concrete parapet and metal rails

This type of barrier has been used for Regular, Medium and Special-performance levels. The barrier consists of a post-and-rail system on top of a concrete parapet.

The heights of the parapet and post are selected based on the requirement of minimum effective height of the barrier.

4.4 Acceptance criteria for bridge traffic barrier types

AS 5100.1:2017 specifies that only barriers which have been demonstrated to the relevant jurisdiction to have acceptable vehicle/barrier interaction performance shall be used. This performance shall be demonstrated by either:

- full-scale crash tests
- methods based on crash-tested barriers used elsewhere
- methods based on barriers that can be geometrically and structurally evaluated as equivalent to a crash-tested system
- for a barrier that has the same details as those of an existing barrier, a method based on an evaluation of the existing barrier's performance, to the approval of the relevant authority.

Crash testing requirements, including safety provisions for each performance level, shall be in accordance with the requirements of the MASH, or other appropriate standards, as determined by the relevant authority. Section 10 provides further detail on the process and requirements of full-scale crash testing.

The Austroads Safety Barrier Assessment Panel (ASBAP) is a panel which was developed to assess the crashworthiness and suitability of road safety barriers, systems and devices for deployment on roads. The Panel evaluates the barriers in accordance with AS/NZS 3845, which utilises the MASH requirements.

As it is unlikely that physical crash tests have been conducted for all bridge barriers in Australia, the barrier designs that are geometrically and structurally equivalent to crash-tested systems should be selected.

In addition to fulfilling the above requirements, the alternative barrier shall be designed in accordance with the design methods presented in this document and to the appropriate performance specification presented in Section 3.

5. Barrier Design

A general layout including bridge barrier, bridge approach barrier and transition is shown in Figure 2.1.

This section provides suggested procedures and detailed guidance for the structural design of bridge barriers located in the area 'A' in Figure 2.1 where the bridge abutments consist of spill-through earth batters. For bridges comprising vertical wall or near vertical wall abutments, such as reinforced soil walls, and where such walls extend parallel to the direction of traffic, area 'A' in Figure 2.1 shall extend to the approach ends of the walls. Where earth embankments are retained by vertical wall or near vertical wall structures, such structures shall be considered no different to a bridge structure when designing traffic barriers, and traffic barriers on such structures shall be designed as for traffic barriers in area 'A' in Figure 2.1.

5.1 Barrier performance specifications

Table 3.1 shows the link between the barrier performance levels with the containment requirements and the required design loads specified in the equivalent MASH test levels.

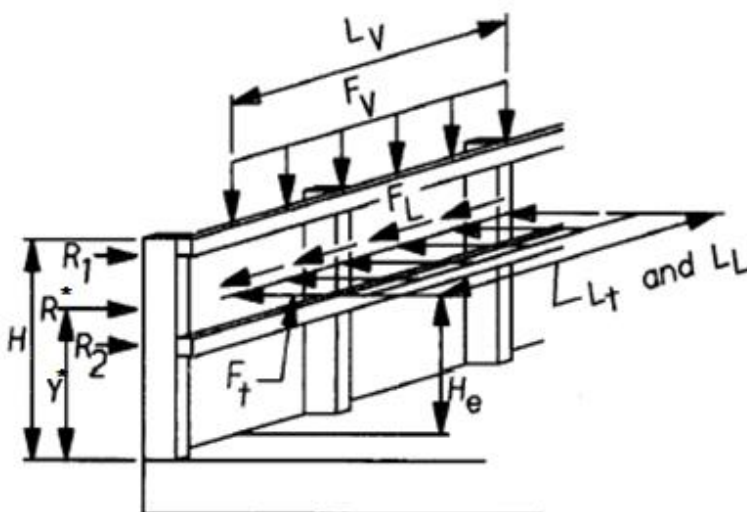
5.2 Structural design criteria

The ultimate design loads and load distribution lengths for all performance levels as defined by AS 5100.2:2017 are given in Table 5.1 and illustrated in Figure 5.1.

Two design checks shall be performed to verify the structural adequacy of the barrier, including:

- Structural adequacy: The barrier total transverse resistance, R^* , shall be greater than or equal to the transverse loads F_t specified in Table 5.1.
- Barrier height: The effective height Y^* of the barrier shall be greater than or equal to the minimum effective height, H_e specified in Table 5.1.

Figure 5.1: Barrier design forces



Source: Based on AASHTO (2020).

Table 5.1: Traffic barrier design loads, contact lengths and effective heights

Barrier performance level	Unit	Low	Regular	Medium	Special	
		TL-2	TL-4	TL-5	TL-6	> TL-6
Ultimate transverse outward load (F_t)	kN	150	300	600	750	1,200
Ultimate longitudinal or transverse inward load (F_L)	kN	50	100	200	250	400
Ultimate vertical downward load (F_v)	kN	22	100	300	375	600
Vehicle contact length for transverse loads (L_t) and longitudinal loads (L_L)	mm	1,100	1,200	2,400	2,400	2,500
Vertical contact length for vertical loads (L_v)	mm	5,500	6,000	12,000	12,000	15,000
Minimum effective height H_e	mm	600	900	1,200	1,800	1,500*

* Typical value – the minimum effective height for the special performance levels shall be subject to special investigation, consistent with AS 5100.1:2017.

Source: Adapted from AS 5100.2:2017.

The following should be noted:

- The design of a barrier system using Table 5.1 shall include a detailed analysis, such as a yield line analysis for a concrete barrier or an inelastic plastic moment analysis for a steel post and rail barrier. Load combinations to be considered include:
 - transverse and longitudinal loads applied simultaneously
 - vertical loads only.
- The loads are applied uniformly over the relevant specified contact lengths.

The values in Table 5.1 were derived based on information in AS 5100.2:2017.

The design procedures in Appendix A13 in AASHTO (2020) are not applicable to traffic railings mounted on rigid structures, such as retaining walls or spread footings, when the cracking pattern is expected to extend to the supporting components.

5.2.1 Contact lengths

The lengths for L_t , L_L and L_v relate to the length of significant contact between the vehicle and barrier that has been observed in films of crash tests. The length L_t , and L_L of 1,200 mm for the Regular performance level (TL-4) is the rear-axle tyre diameter of a truck. The length L_t , and L_L of 2,400 mm for the Medium performance level (TL-5) is the length of a prime mover or trailer rear tandem axles (two 1,050 mm diameter tyres, plus 300 mm between them).

5.2.2 Design forces

F_v is the weight of the vehicle lying on top of the bridge barrier and is distributed over the length of the vehicle in contact with the rail, L_v .

Design forces specified for Special performance level barriers, TL-6 and >TL-6, are taken respectively as approximately 25% and 100% greater than those specified for Medium performance level barriers, to provide clearly recognisable steps in containment capacity over those defined for lower performance levels.

5.2.3 Minimum effective heights

The required minimum effective heights of Low, Regular and Medium performance level barriers are based upon theoretical assessments supported by overseas testing and in-service performance, to prevent the relevant vehicles given in Table 14.4 of AS 5100.1:2017 from rolling over these barriers under the specified impact criteria.

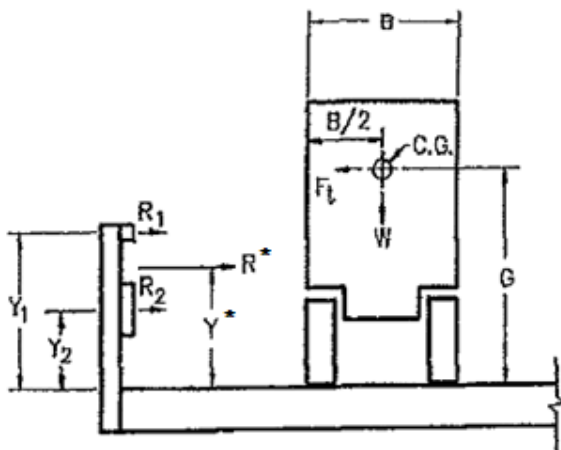
Theoretically, the effective height of the vehicle rollover force, H_e (shown in Figure 5.2) is calculated using Equation 2, with the inputs shown in Figure 5.2.

$$H_e = G - \frac{WB}{2F_t} \quad 2$$

where

- G = height of vehicle centre-of-gravity above bridge deck (mm)
- W = weight of vehicle corresponding to the required test level (N)
- B = out-to-out wheel spacing on an axle (mm)
- F_t = transverse force, specified in Table 5.1, corresponding to the required test level (N)

Figure 5.2: Barrier design diagram



Source: Based on AASHTO (2020).

In relation to the minimum effective height, the following should be noted:

1. The minimum effective height of 600 mm of Low performance level barriers is based upon the tyre height of the test vehicles and other light vehicles.
2. The minimum effective height of 900 mm of Regular performance level barriers is based on satisfactory performance in preventing light test vehicles and the 10-tonne rigid test truck rolling over such barriers, under the impact criteria specified in the crash test.
3. The minimum effective height of 1,200 mm for Medium performance level (TL-5) barriers – a concrete barrier with this minimum effective height was successfully crash-tested under the impact criteria specified in AS 5100, see Section 10.
4. The minimum effective height shown in Table 5.1, for Special performance level (TL-6) barriers is the height required to engage the side of the tank as determined by a crash test.

5. For Special performance level barriers, the effective height of 1,500 mm is aimed at containing the tray of heavy high centre-of-gravity van type semi-trailers. This is to prevent them from rolling over the barrier.
6. In order to prevent a Special performance level heavy tanker-type semi-trailer from rolling over the barrier, a greater effective height, in the order of 1,800 mm to 2,000 mm may be required for a combination parapet. Hence, in the case of a very high-risk situation, an individual benefit cost assessment is still required.
7. In the case of other Special performance level barriers, the appropriate effective height shall be determined and specified by the relevant jurisdiction.

5.3 Concrete parapets

5.3.1 Barrier geometry

Based on AS 5100.1:2017, the F-shape and single slope concrete barrier profiles shall be used as the basis for the parapet, and combined parapet and horizontal rail traffic barriers, unless approved otherwise by the relevant authority. For these profiles, the height of the barrier shall be determined by the required barrier performance level.

5.3.2 Design process

The method used in this section has been adopted from AASHTO (2020), which is based on yield line failure analysis.

It is assumed in this analysis that the yield line failure pattern occurs within the parapet only and does not extend into the deck. This means that the deck must have sufficient resistance to force the yield line failure pattern to remain within the parapet. If the failure pattern extends into the deck, the equations for resistance for the parapet are not valid.

The analysis is also based on the assumption that sufficient longitudinal length of parapet exists to result in the yield line failure pattern shown in Figure 5.3 or Figure 5.4. For short lengths of parapet, a single yield line may form along the junction of the parapet and deck. Such a failure pattern is permissible, and the resistance of the parapet should be computed using an appropriate analysis.

An example of a parapet barrier that was designed using the yield line method, and where a rigid truck towing a trailer struck the barrier at a high angle of impact on Melbourne's Bolte Bridge on 17 May 2013, and the barrier deformed demonstrating yield line deformation, is described by Grzebieta and Rechnitzer (2013).

This analysis assumes that the negative and positive wall resisting moments are equal and that the negative and positive beam resisting moments are equal.

The measurement of system resistance of a concrete barrier is $R^* = R_w$ in this case, which is compared to the loads in Table 5.1 to determine structural adequacy. The flexural resistances, M_b , M_w and M_c , (Figure 5.3), are related to the system resistance R_w through the yield line analysis. R_w is the 'nominal resistance' because it is compared to the 'nominal load' given in Table 5.1.

The procedure includes the following steps:

- Step 1: Calculate the concrete capacities of the various components of the barrier, which includes:
 - M_b – flexural capacity of the beam at the top of the wall about the beam's vertical axis (if the beam exists)
 - M_w – flexural capacity of the wall about the vertical axis
 - M_c – flexural capacity of the wall about an axis parallel to the bridge's longitudinal axis.

- Step 2: Calculate the critical length of the yield line failure pattern L_c .
- Step 3: Calculate the barrier's resistance R_w .
- Step 4: Check the adequacy of the barrier.

Two impact cases may occur. Firstly, impact within the wall segment (Figure 5.3) and secondly, impact near the end of the wall segment (Figure 5.4).

Impact within wall segment

The barrier resistance R_w and the critical wall length L_c are calculated as (Equation 3 and Equation 4):

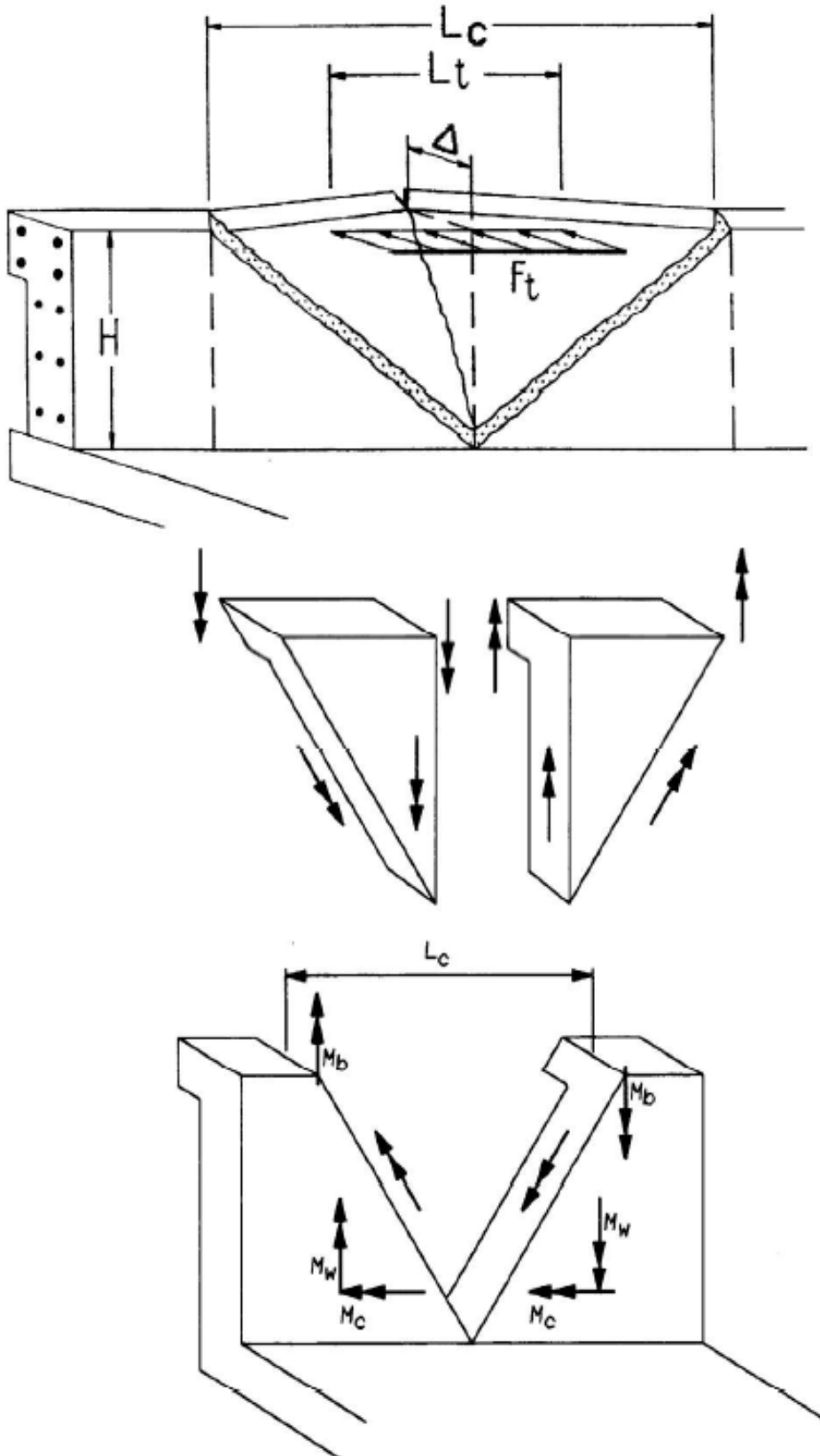
$$R_w = \left(\frac{2}{2L_c - L_t} \right) \left(8M_b + 8M_w + \frac{M_c L_c^2}{H} \right) \quad 3$$

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2} \right)^2 + \frac{8H(M_b + M_w)}{M_c}} \quad 4$$

where

- H = height of wall (mm)
- L_c = critical wall length over which the yield line mechanism occurs (mm)
- L_t = longitudinal length of distribution of impact force F_t specified in Table 5.1 (mm)
- R_w = total transverse resistance of the barrier (N)
- M_b = additional flexural resistance of beam about the vertical axis in addition to M_w , if any, at top of wall (N.mm)
- M_w = flexural resistance of the wall about the vertical axis (N.mm/mm)
- M_c = flexural resistance of cantilevered wall about an axis parallel to the longitudinal axis of the bridge, per metre strip (N.mm/mm)

Figure 5.3: Failure mechanism for impact within wall segment



Source: AASHTO (2020).

Impacts near end wall segment

In this case, the barrier resistance R and the critical wall length L_c are calculated as (Equation 5 and Equation 6):

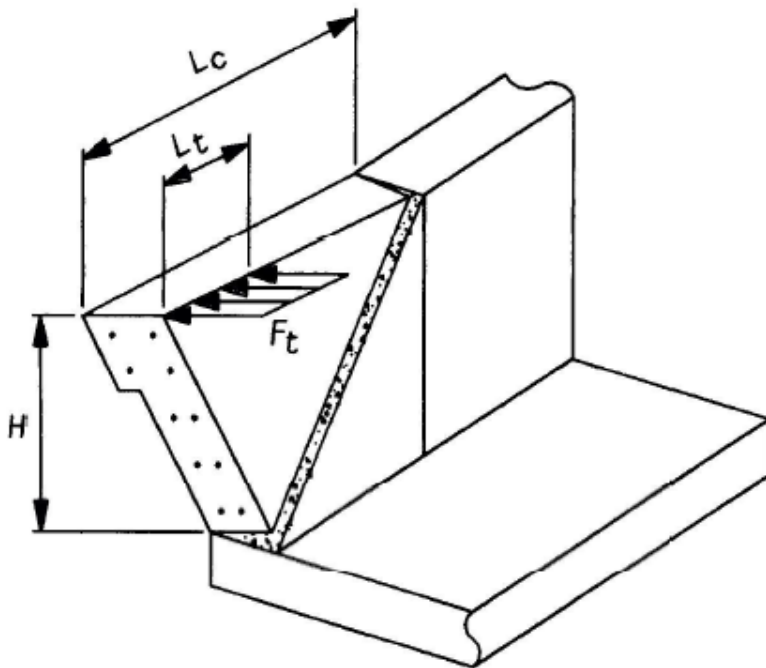
$$R_w = \left(\frac{2}{2L_c - L_t} \right) \left(M_b + M_w + \frac{M_c L_c^2}{H} \right) \quad 5$$

$$L_c = \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2} \right)^2 + H \frac{(M_b + M_w)}{M_c}} \quad 6$$

where

- H = height of wall (mm)
- L_c = critical wall length over which the yield line mechanism occurs (mm)
- L_t = longitudinal length of distribution of impact force F_t , specified in Table 5.1 (mm)
- R_w = total transverse resistance of the barrier (N)
- M_b = additional flexural resistance of beam about the vertical axis in addition to M_w , if any, at top of wall (N.mm)
- M_w = flexural resistance of the wall about the vertical axis (N.mm/mm)
- M_c = flexural resistance of cantilevered wall about an axis parallel to the longitudinal axis of the bridge, per metre strip (N.mm/mm)

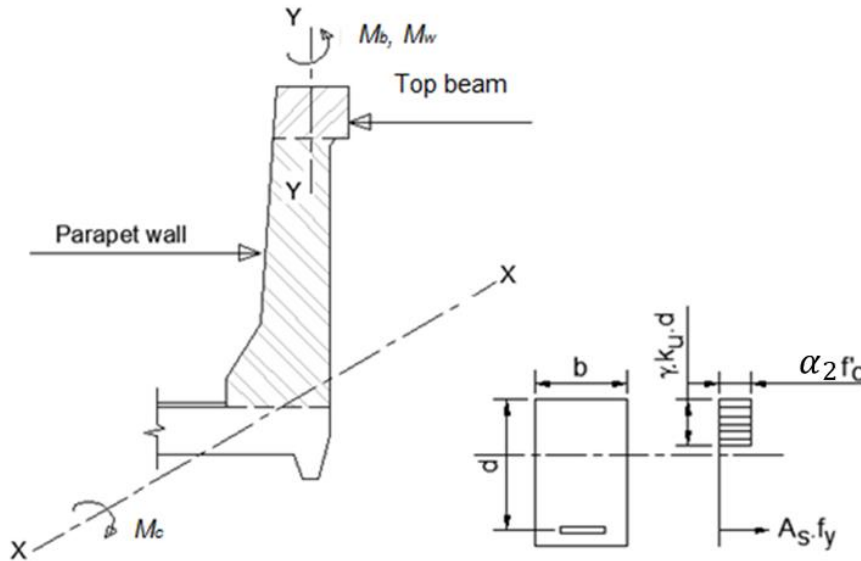
Figure 5.4: Failure mechanism for impact near end wall segment



Source: AASHTO (2020).

The flexural capacity of the barrier components, M_b , M_c , and M_w can be calculated as shown in Figure 5.5 using Equation 7 (based on AS 5100.5:2017 for a rectangular concrete section with tensile steel).

Figure 5.5: Calculation of M_b , M_w and M_c



$$M = \phi M_u = \phi A_s f_y d \left(1 - 0.5 \frac{A_s f_y}{\alpha_2 b d f'_c} \right)$$

7

where

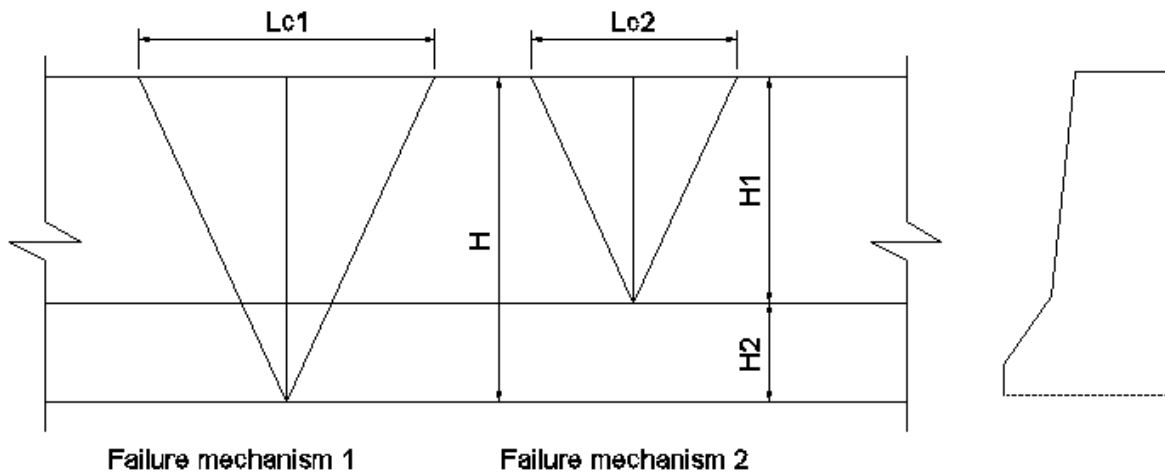
- ϕ = capacity reduction factor, taken from AS 5100.5:2017 for concrete members
- A_s = total cross-section of reinforced bar in the beam (mm^2)
- f_y = minimum yield strength of steel reinforcement (N/mm^2)
- α_2 = $1.0 - 0.003 f'_c$ (within the limits of $0.67 \leq \alpha_2 \leq 0.85$) (AS 5100.5:2017)
- d = effective height of the beam section (mm)
- b = effective width of the beam section (mm)
- f'_c = minimum yield strength of concrete

The following should be noted:

- For use in the above equations, M_c and M_w should not vary significantly over the height of the wall. For other cases, a rigorous yield line analysis should be used.
- Where the width of the concrete wall varies along the height, M_c should be taken as the average of its values along the height of the wall.
- If the parapet length between joints is less than L_c then L_c must be set to the actual length.
- The yield line analysis shown in Figure 5.3 or Figure 5.4 includes only the ultimate flexural capacity of the concrete component. Stirrups or ties should be provided to resist the shear and/or diagonal tension forces.

- Reinforcing steel for concrete barriers shall have sufficient embedment length to develop the yield strength.
- The ultimate flexural resistance of the bridge deck or slab should be determined recognising that the deck is also resisting a tensile force, caused by the components of the impact forces, F_t .
- For F-shape barriers and the like, 2 possible failure mechanisms should be taken into account, as shown in Figure 5.6.

Figure 5.6: Possible failure mechanisms for F-shape barriers and the like



5.3.3 Design of bridge cantilevered deck slab

According to AS 5100.2:2017, the loads transmitted to the bridge deck shall be determined from the results of load testing or ultimate strength analysis of the barrier system using the loads given in Table 5.1.

Sections on the deck slab that should be checked include (Figure 5.7):

- Section A-A at inside face of parapet
- Section B-B at the far edge of the overhang
- Section C-C at the design section in the first interior span.

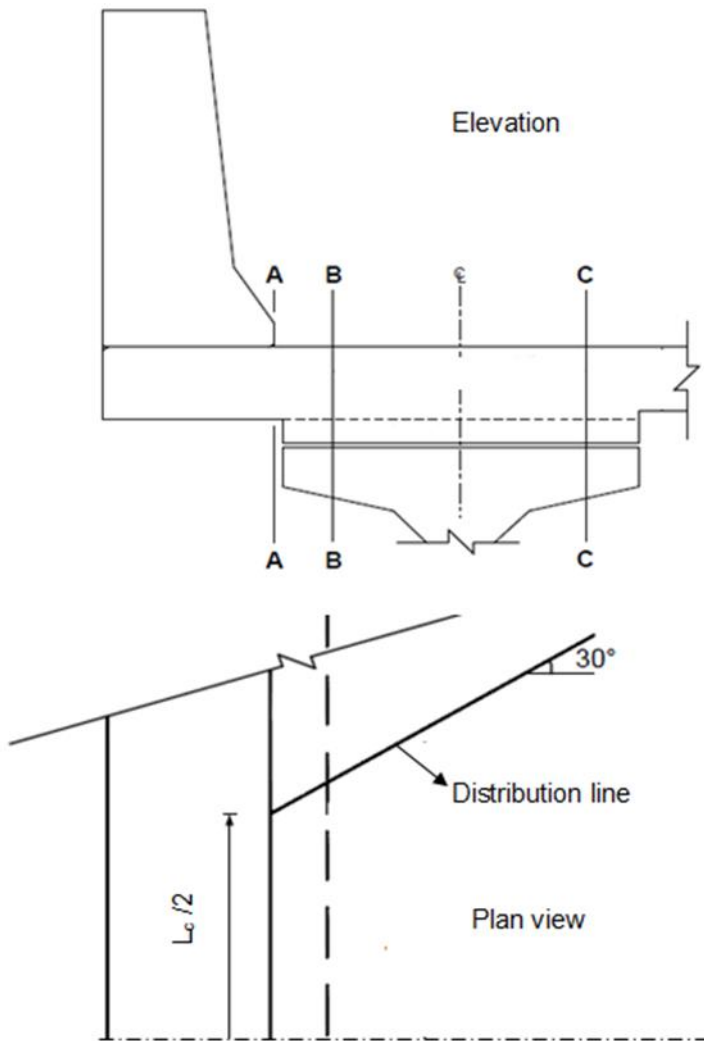
For the design of deck cantilevers, AS 5100.2 requires the design ultimate capacity of the deck to be a minimum of 1.1 times the design ultimate capacity of the barrier connection to the deck. This is to ensure that the bending resistance of the deck slab is larger than the bending resistance of the parapet, thus ensuring failure occurs first in the barrier.

The barrier impact loads and traffic loads on the deck need not be applied simultaneously when designing the deck. For barrier impacts, the following 2 design cases shall be considered:

- Design case 1: Permanent loads + the transverse and longitudinal forces specified in Table 5.1.
- Design case 2: Permanent loads + the vertical forces specified in Table 5.1.

For design case 1, the deck slab shall be designed to resist a bending moment of $1.1 \times M_c$ acting coincident with the axial tensile force $1.1 \times T$. The axial tensile force T is calculated using Equation 8.

Figure 5.7: Design sections in the overhang region



$$T = \frac{R_w}{(L_c + 2H)}$$

8

where

- R_w = parapet resistance (N)
- L_c = critical length of yield line failure pattern (mm)
- H = height of wall (mm)
- T = tensile force per unit of deck length (N/mm)

In the above equation, the tensile strength of the deck slab is calculated based on the actual resistance of the parapet R_w , which ensures that the deck overhang region is designed to have a resistance larger than the actual resistance of the concrete parapet. For tensile forces, a 45° angle for load distribution is assumed, thus the length of the deck slab carrying the tensile load T is $L_c + 2H$.

At the inside face of the parapet, the collision forces are distributed over a distance L_c for the moment and $L_c + 2H$ for axial force. It is reasonable to assume that the distribution length will increase as the distance from the section to the parapet increases.

The value of the distribution angle is not stated in the specifications and is determined using engineering judgment. Distribution angles between 30° and 45° have been used to calculate the increase in the distribution length, which is acceptable. This distribution angle can be used for both sections B-B and C-C in Figure 5.7. An angle of 30° is preferable as it is more conservative.

For design case 2, the design of the deck slab shall be based on the cantilevered portion of the deck. However, for concrete parapets, the case of vertical collision never controls.

If the deck overhang capacity is less than that specified, the yield line failure mechanism for the parapet may not develop as shown in Figure 5.3 and Equation 3, and Equation 4 will not be correct.

The crash testing program is oriented toward survival, not necessarily the identification of the ultimate strength of the barrier system. This could produce a barrier system that is significantly overdesigned, leading to the possibility that the deck overhang is also overdesigned.

5.4 Post-and-rail barriers

5.4.1 Barrier geometry

For traffic barriers with post and horizontal rail components, AS 5100.1:2017 requires the application of the following geometric requirements (Figure 5.8).

The total depth of longitudinal barrier components, including longitudinal rails and any kerb or parapet section above the reference surface, in contact with the vehicle, ΣA , shall not be less than 25% of the height of the barrier (H). The height of the barrier (H) is measured from the reference surface to the upper surface of the top vehicle contact rail.

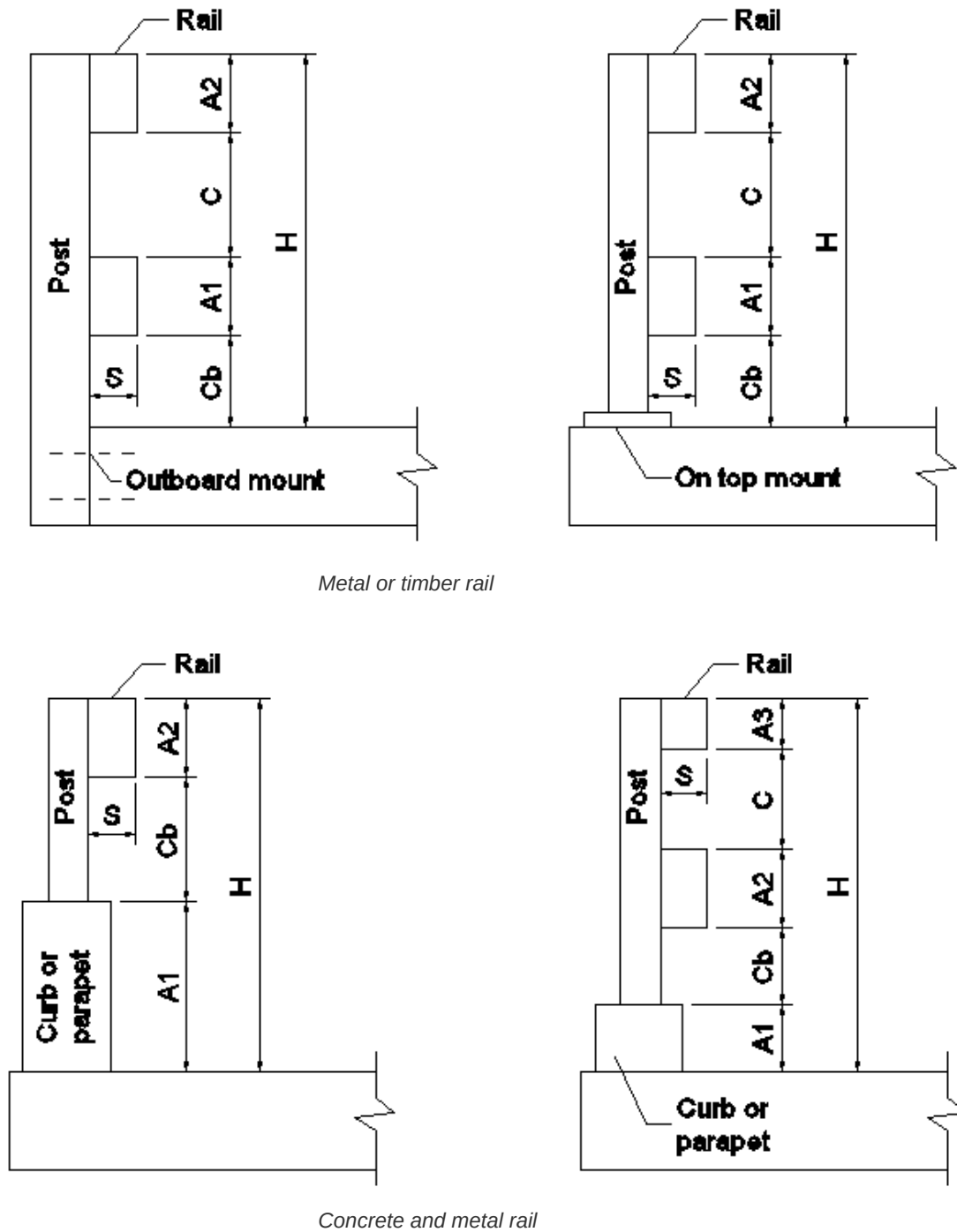
The clear vertical opening (c_b) below the lowest rail shall be not greater than 380 mm. The clear vertical opening between rails (c) or between the top of a concrete barrier and a horizontal rail ($A_1, A_2 \dots$) shall be not greater than 380 mm. The vertical clear opening between rails is the distance between square or rectangular shapes and the extreme traffic-side face of curved or circular shapes. Consideration should be given to providing a kerb to prevent objects falling off the bridge.

Unless approved otherwise by the relevant jurisdiction, posts shall be set back from the traffic face of the traffic rails by a minimum of:

- 100 mm for $\Sigma A/H \geq 0.5$
- 200 mm for $\Sigma A/H \leq 0.3$.

Linear interpolation shall be used where $\Sigma A/H$ is between 0.3 and 0.5.

Figure 5.8: Post and rail barriers



The post setback, S , is defined as the horizontal distance between the traffic faces of the railing and the traffic faces of plane-faced posts or the centre of circular posts.

The traffic faces of all traffic rails shall be within 25 mm of a vertical plane through the face of the rail closest to the traffic. Rails further back than 25 mm or centred lower than 380 mm above the reference surface shall not be considered as traffic rails for the purpose of resisting the design load specified in AS 5100.1:2017.

5.4.2 Design process

It is suggested that an inelastic analysis be used for design of a post-and-rail barrier under failure conditions. The post on each end of the plastic mechanism must be able to resist the rail or beam shear. One, 2, or more barrier spans may be involved in a failure mode. Figure 5.9 presents several possible failure modes, including single-span, two-span, and three-span failure modes.

The procedure includes the following steps:

- Step 1: Calculate the ultimate capacities of the posts and rails.
- Step 2: Calculate the combined barrier resistance.
- Step 3: Check the adequacy of the barrier.

Depending on the location of the failure, there may be 2 cases, including failures that involve and do not involve the end post of a segment.

The critical nominal resistance, $R^* = R_r$ in this case, when the failure does not involve the end post of a segment, shall be taken as the lowest value determined from Equation 9 and Equation 10 for various numbers of railing spans, N , as follows:

- For failure modes involving an odd number of barrier spans, N :

$$R_r = \frac{16M_p + (N-1)(N+1)P_p L}{2NL - L_t} \quad 9$$

- For failure modes involving an even number of barrier spans, N :

$$R_r = \frac{16M_p + N^2 P_p L}{2NL - L_t} \quad 10$$

where

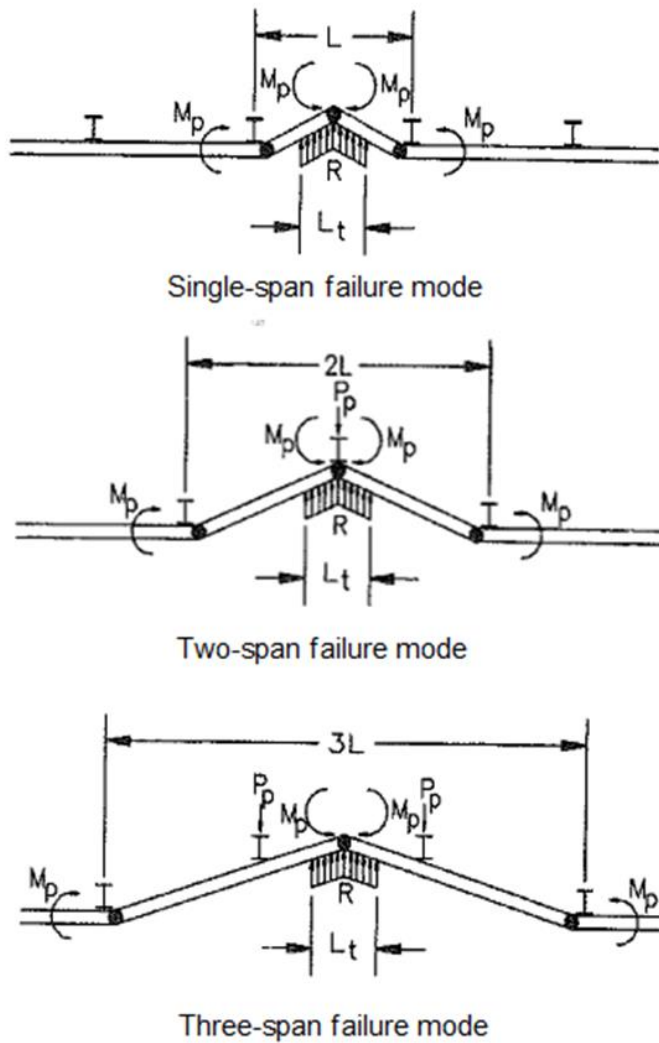
L = post spacing or single span (mm)

M_p = inelastic or yield line resistance of all the rails contributing to a plastic hinge (Nmm)

P_p = shear resistance of a single post, which corresponds to the inelastic resistance of the post and is located Y^* above the deck (N)

R_r = total transverse resistance of the rails and post (N)

Figure 5.9: Possible failure modes for post-and-rail barriers



Source: AASHTO (2020).

The inelastic or yield line resistance of steel post or rail, M_p can be calculated by Equation 11.

$$M_p = \phi f_y S \quad 11$$

where

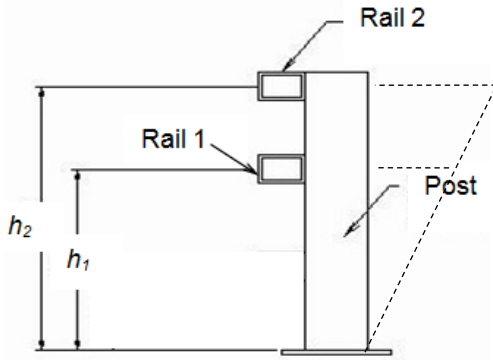
ϕ = capacity reduction factor, $\phi = 0.9$ for steel material

S = plastic section modulus of the rail's cross-section

f_y = yield strength of the rail material

The contribution of each rail to M_p is proportional to the height of the rail (distance from the rail to the base of the post). For example, in Figure 5.10, rail 2 contributes 100% M_{p2} and rail 1 contributes $h_1/h_2 \times M_{p1}$.

Figure 5.10: Contribution of yield line resistance of rails



The shear resistance of the post, P_p is calculated by the following formula, based on the post's inelastic resistance M_{p-post} (Equation 12).

$$P_p = \frac{M_{p-post}}{Y^*} \quad 12$$

where

Y^* = effective height of the barrier

- For impacts at the end of rail segments that cause the end post to fail, the critical rail nominal resistance, R_r , shall be calculated as follows for any number of barrier spans, N (Equation 13).

$$R_r = \frac{2M_p + 2P_p L (\sum_{i=1}^N i)}{2NL - L_t} \quad 13$$

The total transverse resistance of a post-and-rail barrier R^* , shall be greater than or equal to the transverse loads specified in Table 5.1.

The effective height of the barrier, Y^* , is calculated as Equation 14.

$$Y^* = \sum_{i=1}^n \frac{R_i Y_i}{\bar{R}} \quad 14$$

where

n = number of rails

R_i = proportion of the total ultimate resistance taken by the i^{th} rail (N)

$$\bar{R} = \sum R_i$$

distance from the top of the base plate to the i^{th} rail (mm)

Y_i = (AASHTO (2020) defines it as the distance from the bridge deck; however, this does not suit the case where the base plate is fixed on a short upstand)

For multiple rail systems, each of the rails may contribute to the yield mechanism shown schematically in Figure 5.10, depending on the rotation corresponding to its vertical position.

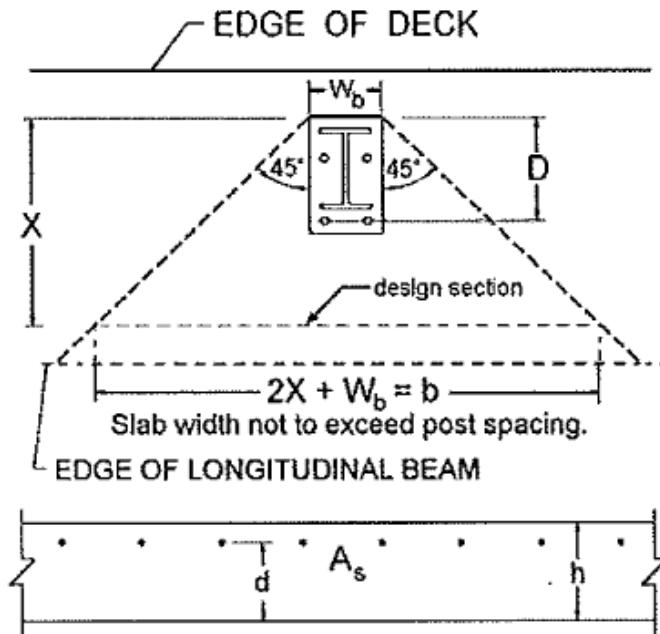
If the total resistance, R_r , of a post-and-rail barrier system with multiple rail elements is significantly greater than the applied load, F_t , then the resistance for the lower rail element(s) used in calculations may be reduced. The reduced value of R_r will result in an increase in the computed value of Y^* . The reduced notional total rail resistance and its effective height must satisfy the requirements in Section 5.2.

5.4.3 Design of bridge cantilevered deck slab

Post-and-rail systems, such as a metal system with wide flange or tubular posts, impose large, concentrated forces and moments on the deck at the point where the post is attached to the deck (Figure 5.11).

Two design cases as described in Section 5.3.3 shall be considered.

Figure 5.11: Effective length of cantilever for carrying concentrated post loads



Source: AASHTO (2020).

For design case 1, the deck shall be designed to resist a moment $1.1 \times M_d$ in N.mm/mm, and a tensile force $1.1 \times T$, in N/mm, calculated as follows (Equation 15 and Equation 16).

$$M_d = \frac{M_{p\text{-post}}}{(W_b + D)} \quad 15$$

$$T = \frac{P_p}{(W_b + D)} \quad 16$$

where

$M_{p\text{-post}}$ = flexural resistance of barrier post (N)

P_p = shear corresponding to $M_{p\text{-post}}$ (N)

W_b = width of base plate of the barrier post (mm)

D = distance from the outer edge of the base plate to the innermost of bolts (mm)

It is noted that Equation 15 is valid when the base plate is fixed directly to the deck and only the deck top steel is designed to resist the tension force. Additional analysis is required if an upstand exists and/or if deck bottom steel is used to resist the tension.

For design case 2, the shear force, P_v and cantilevered moment, M_d are taken as Equation 17 and Equation 18.

$$M_d = \frac{P_v X}{b} \quad 17$$

$$P_v = \frac{F_v L}{L_v} \quad 18$$

in which

$$b = 2X + W_b \leq L$$

where

- F_v = vertical force of vehicle laying on top of rail after impact forces F_t and F_L are over (N)
- L = post spacing (mm)
- L_v = longitudinal distribution of vertical force F_v on top of barrier (mm)
- X = distance from the outside edge of the post base plate to the section under investigation, as specified in Figure 5.11 (mm)

The yield strength of anchor bolts for steel barriers shall be fully developed by bond, hooks, attachment to embedded plates, or any combination thereof.

5.5 Concrete parapet and metal rail

5.5.1 Barrier geometry

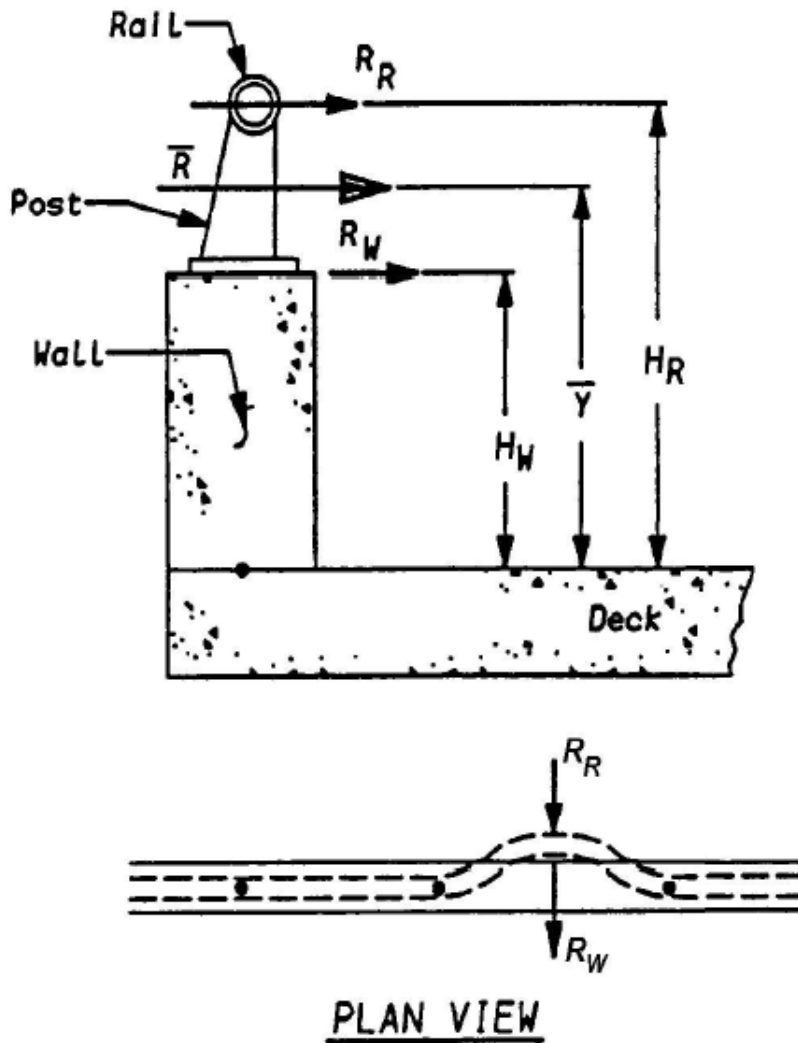
The geometric requirements described in Sections 5.3.1 and 5.4.1 shall also apply to traffic barriers with concrete parapet and metal rails (Figure 5.12).

5.5.2 Design process

The resistance of each component of a combination parapet and rail system shall be determined as specified in Section 5.3 and Section 5.4. The flexural strength of the rail, R_r shall be determined over one span, and over 2 spans. The resistance of the post on top of the wall P_p , shall be determined including the resistance of the anchor bolts or post.

The barrier resistance of the combination parapet and rail shall be taken as the lesser of the resistances determined for 2 failure modes, including impacts at the mid-span of the rail (Figure 5.12) and impacts at the post (Figure 5.13).

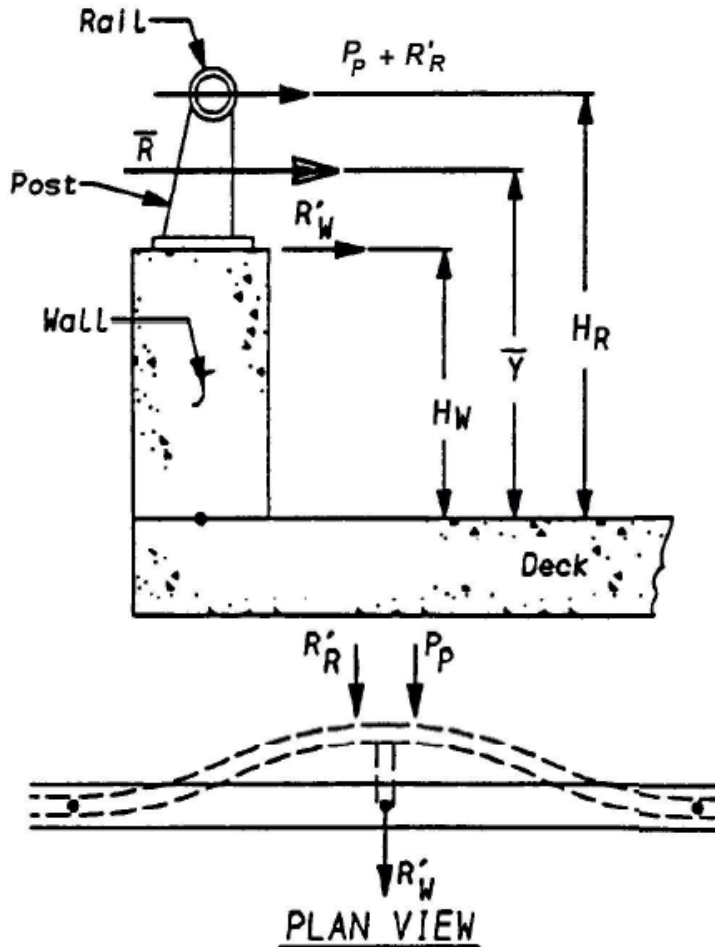
Figure 5.12: Concrete parapet and metal rail – impact at mid-span of rail



Note: Based on AS 5100.1:2017, an F-shape or single slope barrier shall be used as the bases for a combined parapet, unless approved otherwise by the relevant authority.

Source: AASHTO (2020).

Figure 5.13: Concrete parapet and metal rail – impact at post



Note: Based on AS 5100.1, an F-shape or single slope barrier shall be used as the bases for a combined parapet, unless approved otherwise by the relevant authority.

Source: AASHTO (2020).

The procedure includes the following steps:

- Step 1: Calculate the ultimate capacities of the various components, including concrete parapet and rail-and-post.
- Step 2: Calculate the resistance of the combination parapet and rail.
- Step 3: Check the adequacy of the barrier.

Impact at mid-span of the metal rail

Where the vehicle impact is at mid-span of the metal rail, the flexural resistance of the rail, R_r , and the maximum strength of the concrete wall, R_w , shall be added together to determine the combined resultant strength, R^* and the effective height Y^* , taken as Equation 19 and Equation 20.

$$R^* = R_r + R_w \quad 19$$

$$Y^* = \frac{R_r H_r + R_w H_w}{R^*} \quad 20$$

where

R_r = ultimate capacity of the rail over one span (N), calculated by Equation 9

R_w = ultimate capacity of the wall (N), calculated by Equation 3

Y^* = effective height of the barrier (mm)

H_w = height of wall (mm)

H_r = effective height of rails (mm)

Impact at a post

Where the vehicle impact is at a post, the post takes load until the plastic moment of the post is reached. Beyond this point the remaining load has to be transferred by the deflection of post and rails to the adjacent post.

The wall resistance R'_w is the reduced resistance of the wall due to the load the post is taking. This is because in the end, the post load has to be transferred to the concrete.

The maximum resultant strength R^* , located at a height Y^* , shall be taken as the sum of the post capacity, P_p , the rail strength, R'_r , and a reduced wall strength, R'_w as follows in Equation 21, Equation 22 and Equation 23.

$$R^* = P_p + R'_r + R'_w \quad 21$$

$$Y^* = \frac{P_p H_r + R'_r H_r + R'_w H_w}{R^*} \quad 22$$

in which 23

$$R'_w = \frac{R_w H_w - P_p H_r}{H_w}$$

where

P_p = ultimate transverse resistance of post (N)

R'_r = ultimate transverse resistance of rail over 2 spans (N)

R_w = ultimate transverse resistance of walls (N)

R'_w = capacity of wall, reduced to resist post load (N)

AASHTO (2020) defines P_p as the resistance of the post located Y^* above the deck; however, it should be located at $(H_r - H_w)$ above the top of the parapet and is calculated as Equation 24.

$$P_p = \frac{M_p}{H_r - H_w} \quad 24$$

It should also be recognised that a maximum effective height, Y^* , equal to the centroid rail height, H_r , could be obtained, but at reduced resultant strength, R^* , equal to the post capacity, P_p , and rail capacity, R'_r , only.

The total transverse resistance of a combination concrete wall and metal rail barrier R^* , shall be greater than or equal to the transverse containment loads specified in Table 5.1.

The analysis does not consider impacts near open joints in the concrete wall or parapet. The metal rail will help distribute load across such joints. Improved rail resistance will be obtained if the use of expansion joints is minimised.

For impact near the end of barrier segments, the resistance may be calculated as the sum of the wall resistance, calculated using Equation 5, and the metal rail resistance over one span, calculated using Equation 13.

5.5.3 Design of bridge cantilevered deck slab

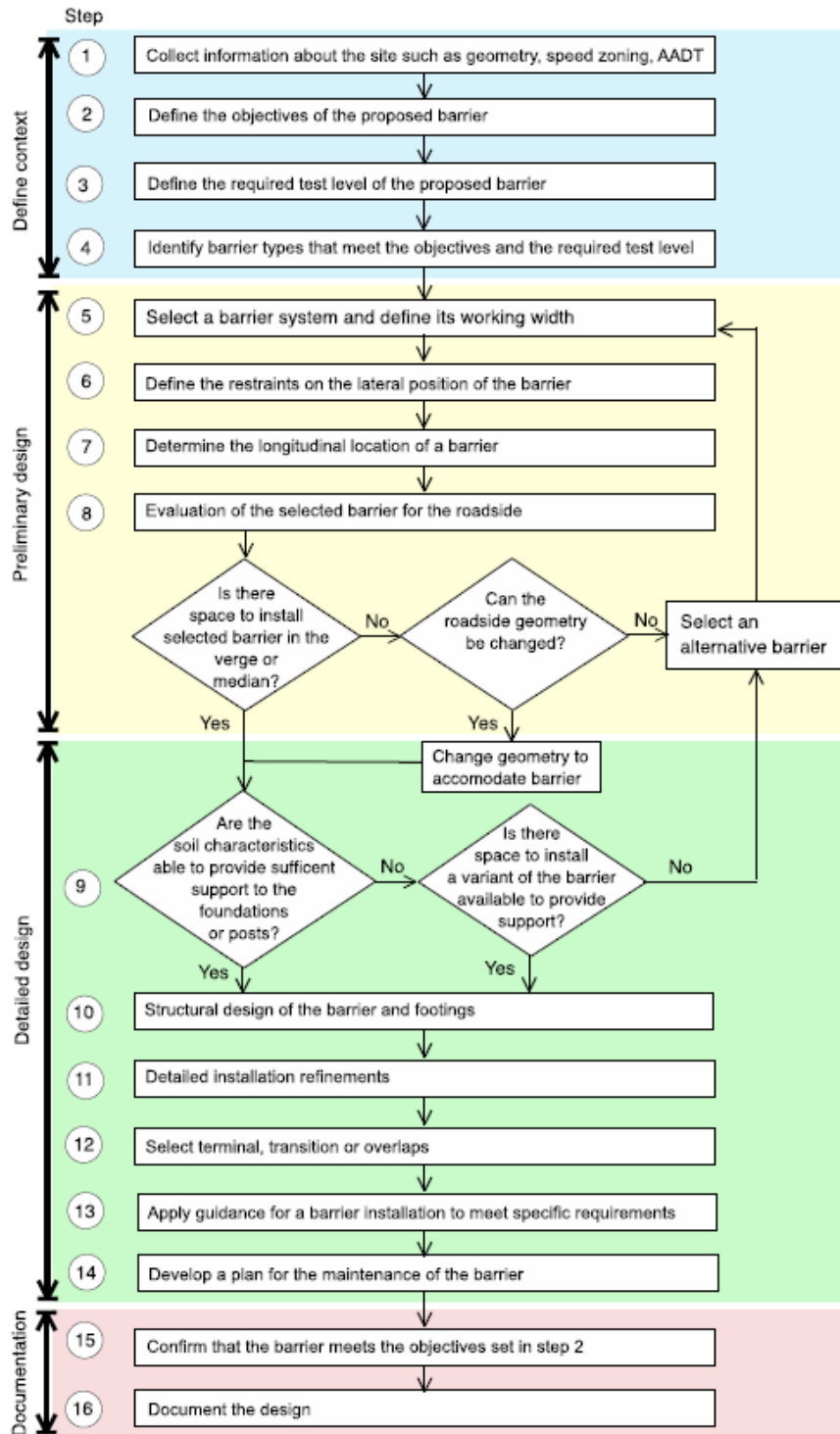
The procedure described in Section 5.3.3 shall be applied.

5.6 Median barrier

Barriers located on a bridge to separate traffic (median barrier) should be capable of being impacted from either side and prevent vehicles from entering the opposing direction. This generally means that the dynamic deflection of the barrier on impact is considered, i.e. a median shoulder should be provided to contain the deflection. As there is no discernible difference between the function of a median barrier that separates traffic when on a bridge or a road, the design and performance level of these barriers should be designed in accordance with Austroads *Guide to Road Design Part 6: Roadside Design, Safety and Barriers* (Austroads 2024), the process and relevant sections within the Guide, which includes median barriers, is provided in Figure 5.14. Deflection and working widths of barriers are discussed in Section 5.5.2 and Section 6.7 of Austroads (2024).

Consideration should be given to the requirements to secure the barrier to the bridge structure. This may include the continuation of the barrier over bridge joints, impact loading and transfer to the structure if struck and vehicle roll allowance.

Figure 5.14: Median and verge barrier design process



Source: Austroads (2024).

6. Bridge Approach Barriers

6.1 Introduction

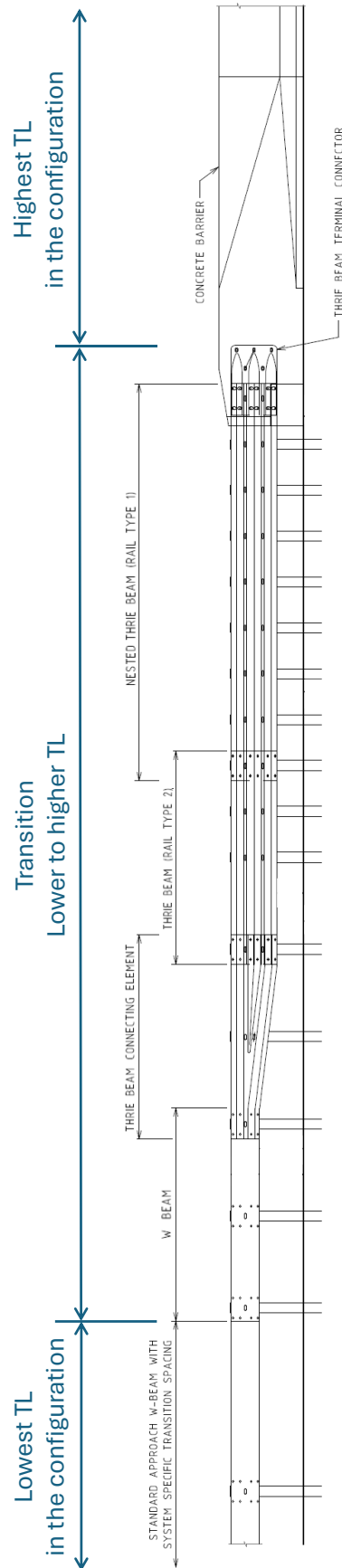
A safety barrier on the approach to a bridge is defined as the 'approach barrier'. An approach barrier can be provided to protect a bridge structure and/or high-risk areas behind a bridge approach (i.e. as per AS 5100.1) or hazards adjacent to, or behind, the bridge approach (as in Austroads 2024).

The length of an approach barrier will be dependent on the function it is providing. An approach barrier should:

- Be of adequate length to protect an errant vehicle from a roadside hazard, bridge parapet/barrier or prevent an errant vehicle from entering a high-risk area behind the approach (AS 5100.1:2017). The required length is determined by a length of need assessment (refer to Section 6.3) and consideration of the hazard or area being protected.
- Provide the same performance level (Test Level) as the bridge barrier for part of, or the full length of the approach barrier. The length of the Test Level (TL) and any changes in TL are determined by AS 5100.1:2017 (refer to Section 6.3.2).
- Provide a transition between different TL (Figure 6.1). This is a section of the barrier that provides a gradual increase in stiffness between a lower level and higher level TL barrier (refer to Sections **Error! Reference source not found.** and 6.4).
- Provide an end treatment (refer to Section 0).

The need to provide an approach barrier was previously identified by considering clear zones. As per the *Austroads Guide to Road Design Part 6* (AGRD Part 6) (Austroads 2024), the clear zone concept is no longer used to determine the need for safety barriers. Should an approach barrier not be warranted as per the guidance in AS 5100.1, the need for an approach barrier can be determined by calculating the Combined Risk Score (Austroads 2024) and comparing that to the relevant Network Roadside Intervention Threshold.

Figure 6.1: Approach barrier example



Source: Victorian Department of Transport and Planning (2021a).

6.2 Current design and specification requirements

6.2.1 Terminology

The following terminologies are used in this guideline:

- Length of need (LON) is the length of a road safety barrier system, excluding leading or trailing terminals, needed to prevent an errant vehicle from impacting a hazard, and represents the length over which a road safety barrier will re-direct an impacting errant vehicle. It is the distance between the leading and trailing points of need – effectively the length of the road safety barrier less terminals.
- Point of need is the start or end of the LON, defining the length over which an errant vehicle is redirected by the road safety barrier and would otherwise strike the hazard if a road safety barrier was not provided. Note that AGRD Part 6 (Austroads 2024) uses the terms 'point of need' and 'points of redirection' interchangeably.
- Right of way (ROW) width is the amount of land reserved or used for transport purposes.

6.2.2 Current AS 5100 requirements

AS 5100.1:2017 specifies that an appropriate transition of compatible stiffness shall be provided for bridge approach barriers.

Transition barriers are specified for bridge approaches with provisions for strength, stiffness, exposed surfaces and end treatments. Strength and stiffness shall vary to provide a transition between any flexible roadside barrier and the rigid or semi-rigid bridge traffic barrier.

A smooth face and tensile continuity shall be maintained throughout. Exposed rail ends, posts and sharp changes in the geometry of the barrier components, kerbs, and the like, shall be avoided or transitioned out with a maximum taper of 1 in 10 for the barrier components, and a maximum taper of 1 in 20 for kerb discontinuities (Clause 14.6.3, AS 5100.1:2017).

It is specified in AS 5100.1:2017 that the selection of a barrier performance level for bridge approaches and departures be based on the same procedure as for the bridge barriers. The length of each performance level provision shall take into account local factors including, but not limited to, the following:

- the distance and clearance to the ROW boundary as it may affect the risk to occupants of errant vehicles and third parties
- the distance to hazards, including rigid objects and steep descents, as it may affect the risk to occupants of errant vehicles and third parties
- the risk associated with the use of the crossing underneath the bridge and the proximity of that crossing
- the risk associated with the existence of service roads or parallel walkways and the like.

There is no detailed guidance for determining the LON of bridge approach barriers and the length of each performance level barrier on the bridge approach. The principles regarding this are provided in Section 6.3.1 and discussed in detail in AGRD Part 6 (Austroads 2024).

6.2.3 Additional requirements from jurisdictions' guidelines

Victorian Department of Transport and Planning

BTN001 (Victorian Department of Transport and Planning 2023) provides guidelines for bridge approach barriers. These guidelines expand the provisions of AS 5100.1:2017 for the design of bridge approach barrier systems for bridges on arterial road networks, on major freeways and divided highways, and on less heavily trafficked roads. The following requirements are specified in addition to the AS 5100.1:2017 requirements:

- If the bridge off-structure barrier is connected to a roadside barrier, the roadside barrier must be continuous with the bridge off-structure barrier system. A transition from the flexible roadside barrier to the more rigid bridge off-structure barrier must be provided at the junction of the 2 systems. The roadside barrier may overlap the end of the bridge off-structure barrier subject to appropriate allowance for its working width.
- The bridge off-structure barrier performance level is stepped-down incrementally to the point where it ends or joins the roadside barrier.
- The initial section of bridge off-structure barrier adjacent to the bridge is assigned the same performance level as the bridge barrier.
- The total length of bridge off-structure barrier at each side and end of the bridge may be limited to 110 m if the calculated total length is longer.
- If an errant vehicle departing the bridge can impact the bridge barriers on the adjacent bridge, the off-structure barrier on the departure end of the structure must be extended to contain errant vehicles at a 15° divergence angle projected from the adjacent bridge or median side of the traffic.

Transport for New South Wales (TfNSW)

TfNSW provides standard drawing TS 02553 (Transport for New South Wales 2022) for the thrie-beam connection to rigid barrier or bridge end post and reduced post spacing.

TfNSW's *Supplement to Austroads Guide to Road Design* (Transport for New South Wales 2023) specifies the TfNSW's departures from the Austroads Guide (Austroads 2024).

Queensland Department of Transport and Main Roads

For bridge approach barriers, the requirements specified in AS 5100.1:2017 are stated in more detail. These include (Queensland Department of Transport and Main Roads 2024):

- There is no snagging hazard to stop the errant vehicle moving between the 2 types of barriers.
- The entire length of the road barrier, transition and bridge barrier must be smooth and continuous to support an errant vehicle's passage along the carriageway.
- The stiffness of the barrier must ensure pocketing does not occur. Bridge transition barriers are detailed in TMR's standard drawings
- The maximum slope on a transition in height shall be no steeper than 1:10.
- The approach rail shall not direct an errant vehicle into a hazard on the bridge, e.g. kerb. Similarly, the exit shall not direct a vehicle onto a hazard. The layout must also address crossover crashes. The approach barrier shall also stop an errant vehicle from travelling down embankments, into creeks or onto a road beneath the bridge.

For the transition section, the publication *Supplement to Austroads Guide to Road Design Part 6: Roadside Design, Safety and Barriers* (Department of Transport and Main Roads 2023a) discourages the direct connection of w-beam to permanent concrete barriers and recommends that it should not be accepted for new installations. A transition from w-beam to thrie-beam to concrete is the preferred method of interfacing w-beam to a permanent concrete barrier system in all situations.

For road/rail interface barriers, the technical specifications *CIVIL-SR-007* (Queensland Rail 2022) shall be applied.

Various types of end treatments are listed in the publication *Accepted Road Safety Barrier System and Devices* (Queensland Department of Transport and Main Roads 2023b), including ET-SS, MSKT, MAX-Tension, Trend Median Terminal, QuadGuard M10, Smart Cushion, Universal TAU-M, Hercules, and others. The selection of an appropriate end treatment depends on factors such as speed limit and barrier application.

New Zealand Transport Agency Waka Kotahi

Waka Kotahi NZ Transport Agency (now NZ Transport Agency Waka Kotahi) (2022) specifies that where the protection level on the approach is less than that provided on the structure, the length of approach transition shall be addressed as the appropriate length of need for the hazard, as per AGRD Part 6 (Austroads 2024).

Main Roads Western Australia

Other jurisdictions' guidelines, such as Main Roads Western Australia (2024b), follow AS 5100.1:2017 design requirements for bridge approach barriers, with supplementary requirements specified in AS/NZS 3845.1:2015 and AASHTO *Roadside Design Guide* (AASHTO 2011) for the transition sections.

6.3 Design procedure

The following design steps are suggested:

- Determine the LON for the bridge approach barrier.
- Select appropriate performance levels for different sections along the required LON.
- Determine transitions between different barrier types.
- Select end treatments.
- Determine structural solutions.

These steps are presented in detail in the following sections.

6.3.1 Determination of length of need

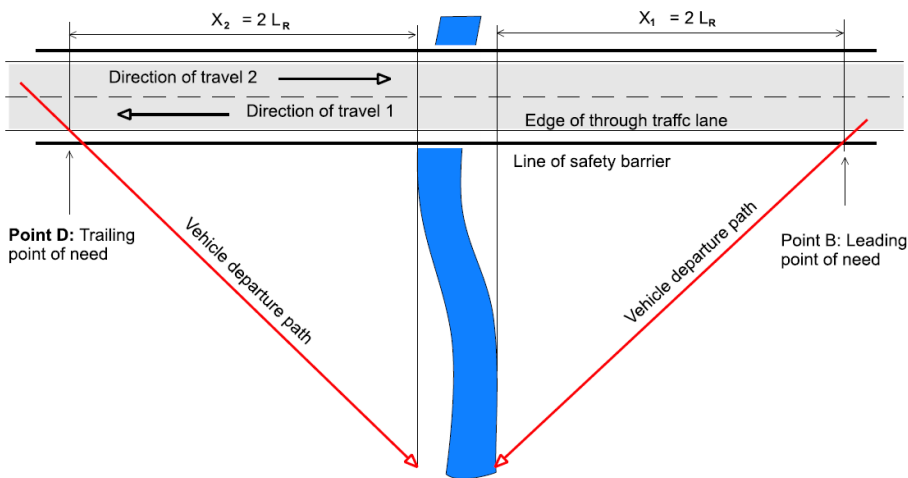
The LON for an approach barrier is the length of barrier required to redirect an errant vehicle and shield hazards, it is defined by the distance between the leading and trailing points of need (as demonstrated in Figure 6.2 to

Figure 6.5. The process in Austroads (2024) is based on vehicle stopping distances (through the run-out length) and vehicles drifting off the road at a small departure angle. Refer to Austroads (2024) for more details of the procedures to calculate points of need.

The LON considers the point of need which identifies the point at which protection is required to prevent a vehicle from exiting the road and impacting the identified hazard, this could include a bridge parapet, bridge barrier or a hazard on the immediate approach to a bridge or behind the approach (i.e. perpendicular to the bridge e.g. a steep embankment, water, trees or roadside infrastructure).

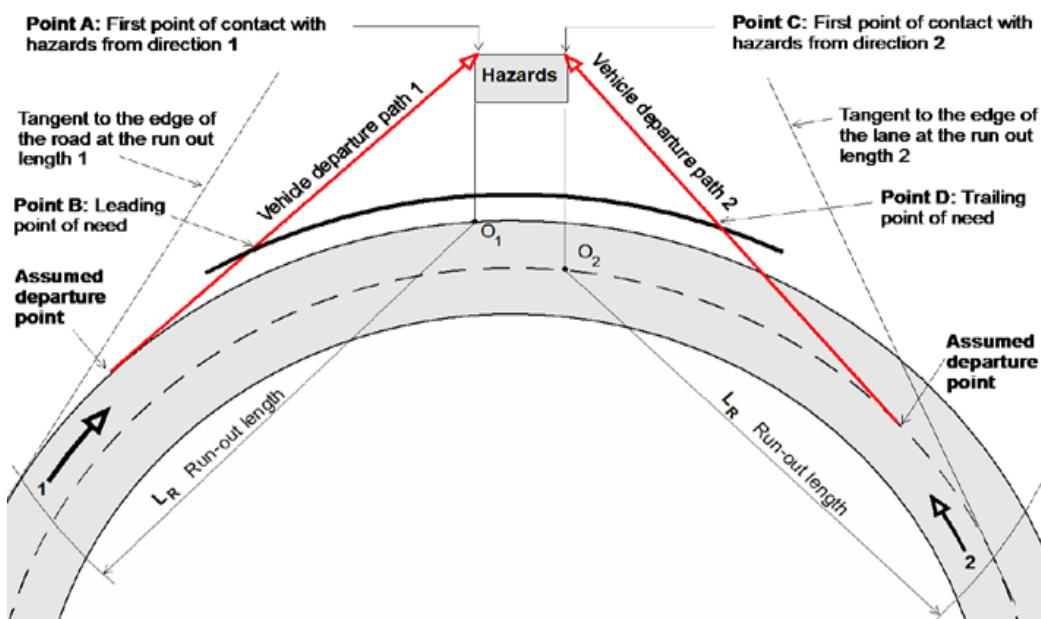
The detailed method to determine the LON is provided in AGRD Part 6 (Austroads 2024). Appendix D.2.2 and AGRD Part 6 (Austroads 2024) also provides guidance for the consideration of alternative departure angles for heavy vehicles.

Figure 6.2: Leading and trailing points of need for hazards extending long distances from the road

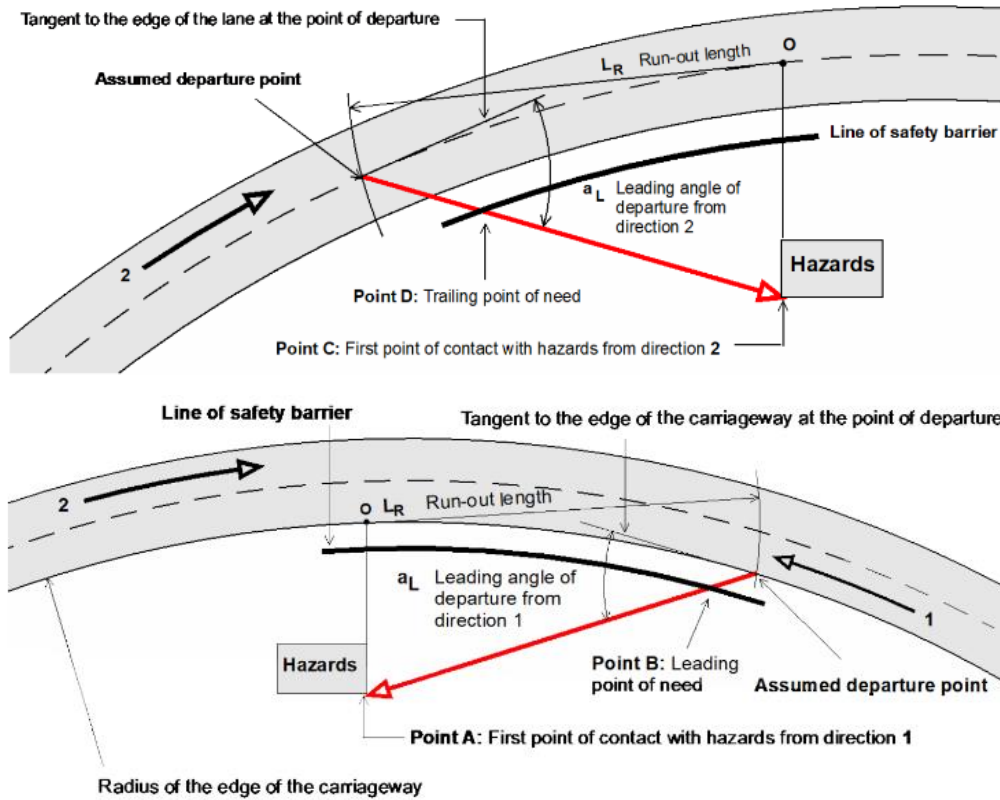


Source: Austroads (2024).

Figure 6.3: Points of need for curved roads with two-way traffic (hazard on outside of small radius curve)

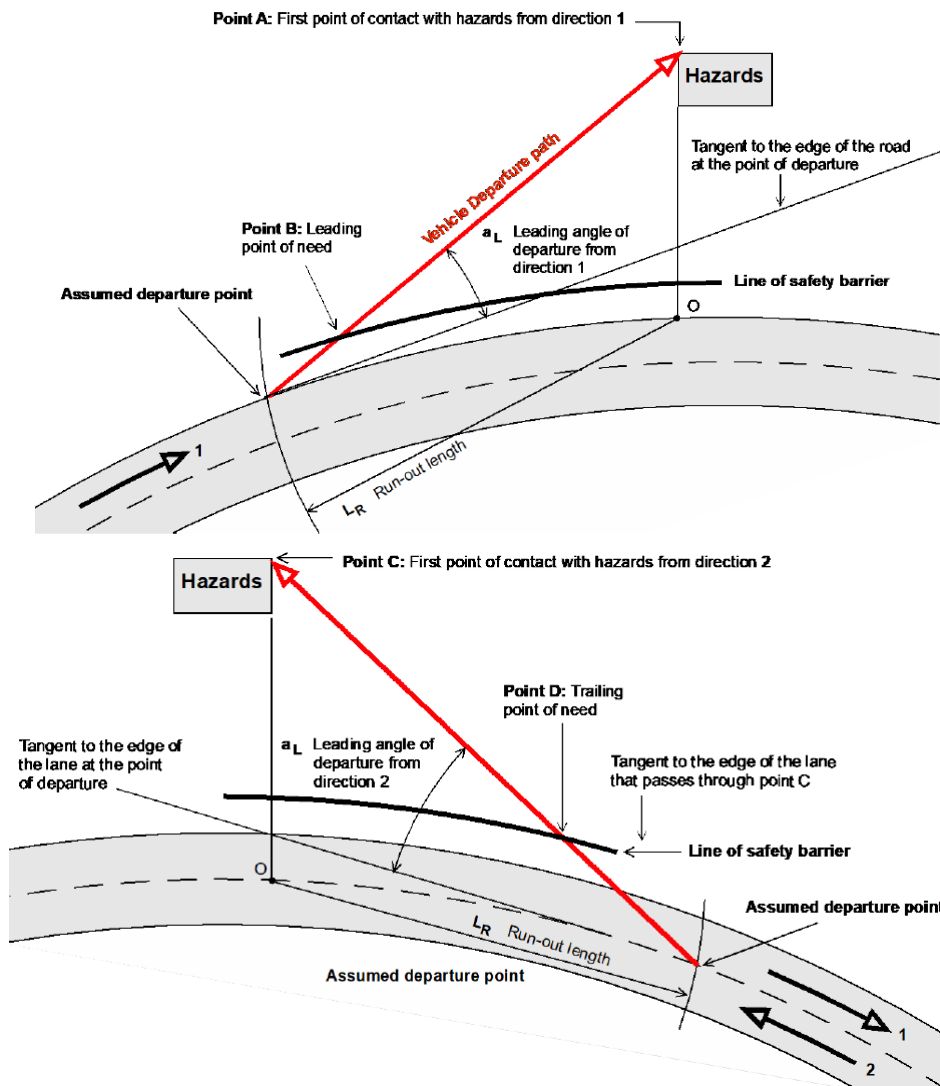


Source: Austroads (2024).

Figure 6.4: Trailing point of need for curved roads with two-way traffic (hazard on inside of curve when $D > L_R$)

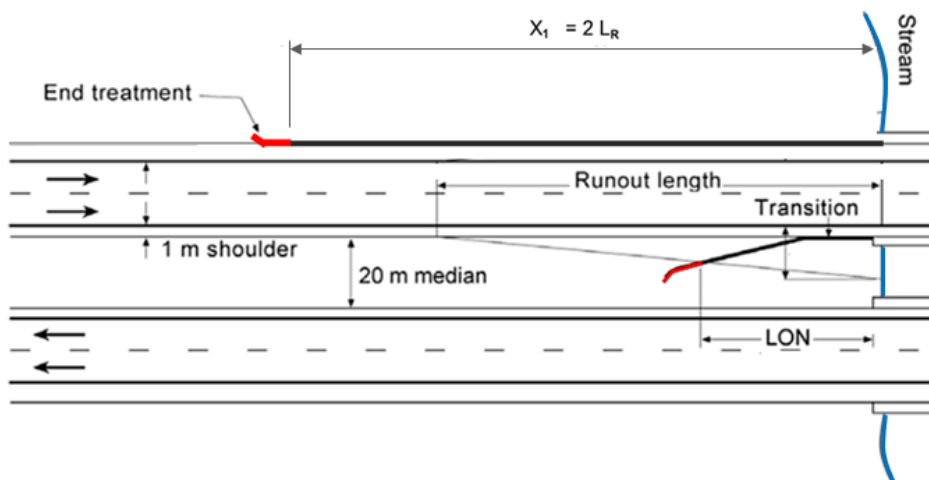
Source: Austroads (2024).

Figure 6.5: Points of need for curved roads with two-way traffic (hazard on outside of large radius curve)



Source: Austroads (2024).

Figure 6.6: Length of need considerations for dual bridges separated by a median



Source: Austroads (2024).

6.3.2 Selection of approach barrier performance level

The performance level of an approach barrier should match the bridge barrier performance level as specified by AS 5100.1, these requirements as summarised in Section 6.2.2.

Based on the possible bridge barrier performance levels, the following should be considered for the approach barriers:

- **No Bridge Barrier:** The requirements in AS 5100.1 provide some logic for identifying when a barrier is not required. However, the need to provide an approach barrier can also be determined by a roadside risk assessment as per AGRD Part 6 (Austroads 2024).
- **Low or Regular:** This performance level is for the containment of light vehicles at low-risk sites. Typically, a transition section between low or regular performance level barrier and the bridge barrier is required. The minimum length of roadside barrier is determined based on the LON.
- **Medium or Special:**
 - The performance level of the first section on the approach adjacent to the bridge is the same as that of the barrier on the structure (bridge barrier). The length of this segment shall be determined based on consideration of the LON to prevent an errant vehicle entering an area that meets the criteria for the requirement to provide medium or special performance level barriers.
 - The LON for medium or special performance barriers may vary dependent on the hazards being protected. For example, the majority of the approach barrier may be a Regular performance level due to an embankment, however a medium or special level performance may be required closer to the bridge barrier to prevent an errant vehicle entering areas with rail lines, high occupancy land use, waterways etc.
 - The bridge approach barriers are considered off-structure barriers. Therefore, the barrier foundation should be taken into consideration.

In existing urban areas or when city side streets and/or footpaths prevent installation of the approach barrier system, consideration should be given to (Colosimo 2011):

- extending the bridge rail and/or approach barrier around the intersection corner in such a manner as to prevent encroachment of a vehicle onto any highway/terrain system below the bridge
- providing traffic-control measures such as speed-restriction or intersection signs
- providing run-off recovery areas
- if the approach barrier is connected to a roadside barrier, it should be continuous with the bridge approach barrier system and only a transition from a flexible (or semi-rigid) to a rigid barrier system if required.

6.3.3 Minimum length and performance level

The following considerations should be given to the minimum length of barrier and sections of performance level on approach barriers:

- **Length of barrier performance level:**
 - The performance level of an approach barrier should match the bridge barrier. The length of this performance level should be such that it extends to the point of need to protect errant vehicles from the identified hazard. When considering the LON for an 'area' as opposed to a point hazard, the lateral extent of the hazard can be used to determine the LON.
 - Consideration should be given to the risk of an errant vehicle (including heavy vehicles) penetrating, vaulting or rolling over the barrier.

- Minimum lengths of different performance level bridge approach barriers:
 - The typical minimum length of barrier needs to relate to the typical manufacturing length (e.g. precast concrete barriers), transition lengths or product lengths (based on manufacturer) to ensure performance.

6.3.4 Minimum length and performance level of a barrier protecting structures in the median

The length of barrier should be such that the LON (determined by the point of need) protects an errant vehicle from the structure in the median and the barrier on the median shoulder of the opposing carriageway (Figure 6.7), alternatively, a barrier that surrounds the structure can be provided (Figure 6.8). The length of need for the example in Figure 6.7 is determined as per Section 6.3.1. Guidance on considering the performance level, length of that barrier and consideration of providing a two-stage approach is provided in Appendix D.2.2. Further guidance for providing barriers to protect bridge supports is provided in Section 7.

Figure 6.7: Length for barrier in the median protecting a structure

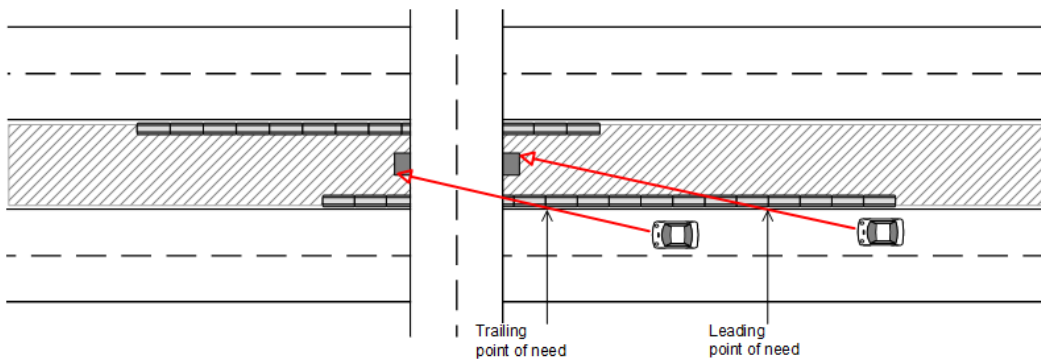
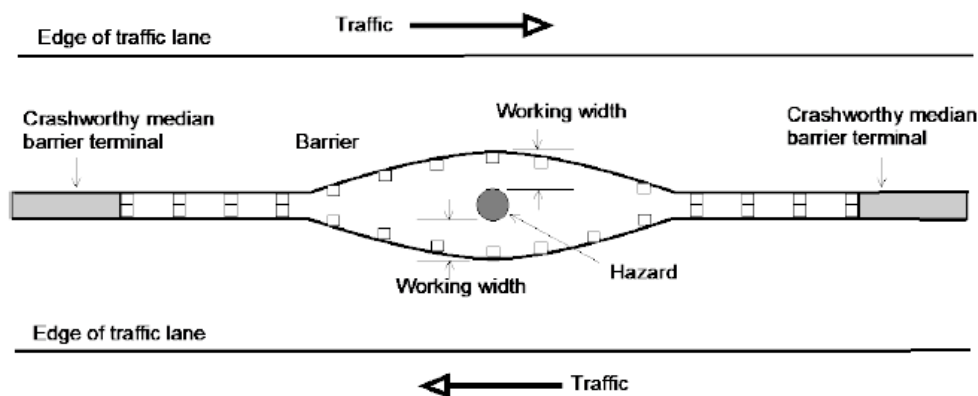


Figure 6.8: An example of a barrier layout for shielding a bridge pier located in a median



Notes:

- 1 Designers should also investigate the use of a crash attenuator to shield the hazard.
- 2 The barrier tapers should be aligned to be consistent with the flare rates, as suggested in Section 6.8.8 of Austroads (2024).

Source: Austroads (2024).

6.4 Transitions between different performance level barriers

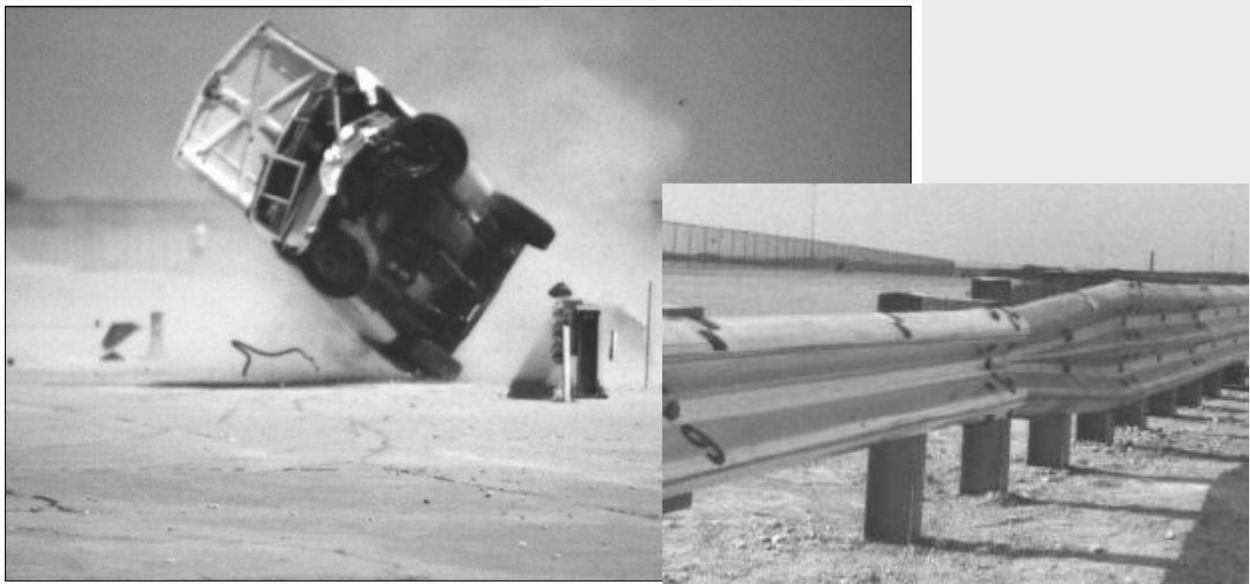
6.4.1 General

In an incident of a vehicle crashing into a roadside barrier on a bridge approach, severe decelerations and the consequent high risk of injury can occur if the transition from a flexible approach barrier to a rigid bridge barrier is not properly detailed and transitioned in strength and stiffness. Such incidents represent a potentially major risk to the occupants of these vehicles.

Transitions are used to provide a safe interface whenever it is necessary to change from one type of barrier to another. A transition section provides a gradual increase in stiffening between the approach barrier systems and/or bridge parapets or barriers so that vehicular pocketing, snagging or penetrations are prevented at any position along the transition. The overlapping of the barriers achieves a similar outcome by providing adequate lateral separation between them.

Case study: Approach barrier

Transitions between different rigidity barriers is important, and a poor transition can cause fatal and serious injury if not properly designed. An example of this is provided in Polivka et al. (2007), which the imagery below shows the rollover of a vehicle caused by an abrupt transition from a W-beam to a more rigid thrie-beam, due to a sudden change in stiffness.



There are 3 broad types of barriers: flexible, semi-rigid and rigid barriers. In general, they have the following features:

- Flexible barriers typically comprise of tensioned wire ropes; however, some W-beam systems are also classified as flexible.
- Semi-rigid barriers typically comprise of horizontal steel beams (e.g. thrie-beam or W-beam).
- Rigid barriers comprise of concrete components.

A satisfactory transition between each barrier type may be achieved by:

- Providing a structurally designed and tested physical connection between the systems: The connections are facilitated through transition sections of barrier that are designed to provide gradually increasing lateral stiffness and hence continuity of protection for vehicles that impact the barrier in the vicinity of the interface. Transitions can be used only between semirigid systems (i.e. steel to steel) or between semi-rigid and rigid systems (i.e. steel to concrete).

- Overlapping the barriers by commencing the more rigid system behind the less rigid system.

In practice, transitions are achieved by the following; however, each system has different requirements, and the transition should be as per the approved method:

- increasing the stiffness of a W-beam system by
 - decreasing the post spacing
 - nesting one rail behind another
 - using another steel section behind the W-beam
- using a heavier rail for the transition.

Transitions should be evaluated using full scale testing. ASBAP has evaluated several transitions both proprietary and non-proprietary. Practitioners should consult jurisdiction's accepted product use for more information.

Overlapping different types of barriers is only possible where adequate space is available to accommodate deflections. An overlap may be used for any barrier systems. It is the only way of achieving a transition from wire rope barrier to a stiffer barrier (see Section 6.14.2 in AGRD Part 6 [Austroads 2024]).

Specially designed barrier sections or connections are used for situations where W-beam is to be connected to thrie-beam, or where either of these semi-rigid barriers is to be connected to a rigid barrier (such as on the approach to bridges that have rigid barriers).

Further detail regarding the requirements for transitions can be found in AGRD Part 6 (Austroads 2024).

6.4.2 Design criteria – physically connected barriers

Several criteria are important when designing a transition section or connection. It is suggested that the principles below be followed where any semi-rigid barrier system is connected to a rigid barrier (AASHTO 2011 and Austroads 2024):

- The connection point of the 2 systems must be as strong as the approach barrier to ensure that the connection will not fail on impact by pulling out. The use of a cast-in-place anchor or through bolt connections is recommended.
- The transition must be designed to minimise the likelihood of snagging an errant vehicle, including one from the opposing lane on a two-way facility.
- When providing a transition section to a bridge railing end it is recommended to flare the bridge railing end behind the approach transition to prevent pocketing on vehicle impact.
- The transition should be long enough to ensure that changes in deflection do not occur over a short distance. The change in stiffness from the less rigid barrier to the more rigid barrier, over the transition length, should increase with a high degree of continuity. This may be achieved by reducing the post spacing, strengthening the rail element or a combination of these techniques.
- As with longitudinal barriers, kerb and slope features must be addressed. The slope between the edge of the road and the barrier should not be steeper than 10:1.
- Drainage features such as kerbs, kerb inlets, raised inlets or open drains should not be constructed in front of barriers or a transition area, as they may initiate vehicle instability and adversely affect the crashworthiness of the barrier or transition.

Designers should not design customised transitions. Designs used on the road should pass MASH testing, be evaluated by ASBAP and be accepted by the road agency.

Transitions, from a stiffer barrier to a more flexible one, are not as critical as those from a flexible barrier to a stiffer one. However, a gradual change in flexibility for the former transition is desirable.

6.4.3 Typical interfaces between barrier types

General

Transitions between semi-rigid and rigid barriers are achieved through stiffening of the steel safety barrier by the use of special sections and connectors, reduced post spacing and nesting (i.e. 2 sections of rail, one inside the other) of the beams.

Wire rope safety barriers (WRSB) are not designed to be connected to semi-rigid or rigid safety barriers or bridge ends. However, WRSB may be transitioned to stiffer barriers provided that the WRSB overlaps the stiffer barrier by an adequate longitudinal distance and the lateral separation is sufficient to accommodate the maximum likely deflection of the WRSB. Such arrangements should enable the 2 systems to work independently while providing continuous shielding of hazards (see Section 6.14.2 in AGRD Part 6 [Austroads 2024]).

Transitions are required to be MASH approved and endorsed by ASBAP. Most state road agencies also have an approved list of barriers and transitions for use.

Where a concrete safety barrier (e.g. F-shape barrier) needs to be transitioned to a vertical or flared back vertical wall, the transition must not be abrupt.

Designers should not design customised transitions. Designs used on the road should pass MASH testing, be evaluated by ASBAP and be accepted by the road agency.

On roads/carriageways with one direction of travel, transitions, from a stiffer barrier to a more flexible one, are not as critical as those from a flexible barrier to a stiffer one. However, a gradual change in flexibility for the former transition is desirable.

Austroads has tested and evaluated a TL-3 transition between a W-beam barrier or a thrie-beam barrier and a permanent concrete barrier. Its design has been incorporated into a jurisdiction's standard drawings and is available as an ASBAP Technical Advice (Austroads 2021b). This design is to be used on bridge approaches. Although this is a TL-3 device, it can still be used to connect to TL-5 concrete barriers and bridge barriers (see Figure 6.9). Other transitions between TL-3 and TL-4 developed by industry are recorded in the jurisdictions' accepted product lists. Only transitions in these lists should be installed and they must comply with manufacturers' specifications, as well as jurisdictions' standard drawings and guidelines. Other designs not included in these lists should not be used.

Figure 6.9: Transition from a W-beam section to a concrete barrier



Source: Austroads (2024).

6.5 End treatments

As specified in AS 5100.1:2017, the end of an approach barrier shall have a crashworthy configuration or be shielded by a crashworthy traffic barrier or impact attenuation device as approved by the relevant authority.

There are several end treatment types, these are broadly known as terminals and crash cushions. Terminals and crash cushions are classified by whether they are 'redirective' or 'non-redirective'. Some crash cushions, such as sand barrels, do not redirect vehicles but capture the errant vehicle, however, these are no longer used (Austroads 2024).

Terminals are classified as being 'gating' or 'non-gating' based on their ability to redirect vehicles that impact along the side of the terminal or crash cushion (Austroads 2024).

The barrier termination shall be designed to ensure acceptable performance on impact at the required performance level for the vehicle(s) under consideration. Any end treatments are in addition to the LON and flaring (see Section 6.8.8 of AGRD Part 6 [Austroads 2024]).

Where rigid bridge barriers are transitioned to a flexible roadside barrier on the bridge approach, the bridge approach barriers will use the same end treatments of the appropriate roadside barriers.

Where bridge barriers end at the bridge approach (do not connect to roadside barriers), the bridge Special performance level barriers should be transitioned to Regular or Medium performance level barriers before connecting to a barrier termination.

6.6 Gore areas

For sites where the required total length of the approach barrier extends to the gore area attenuator, the rigid barrier component shall be extended to the attenuator. For other sites, the extent and type of approach treatment should be based on benefit cost considerations. More detailed information is provided in Sections 5.6.2 and 6.14.2 of AGRD Part 6 (Austroads 2024).

6.7 Freeway and highway ramps and side roads

Barriers in gore areas near to the bridge that include gaps for access or side roads shall be designed to safely contain errant vehicles at all probable impact angles that might result from an errant vehicle leaving the nearest traffic lane at a flare rate determined from Table 6.9 in Austroads (2024).

If the bridge approach barrier is connected to a side road barrier, the side road barrier must be continuous with the bridge approach barrier system. A transition from a flexible to a rigid barrier system at the junction of the 2 systems is then the only requirement.

Appropriate barrier systems, normally the rigid bridge barriers, shall be extended down freeway or major highway entrance and exit ramps a suitable distance to protect against errant vehicles penetrating the freeway or highway trafficked lanes.

6.8 Bridge approach off-structure barrier foundation

The loads transmitted to the barrier base/foundation shall be determined from an ultimate limit states analysis of the barrier system using the loads specified in AS 5100.2:2017.

In the case of foundations for rigid barrier systems, the capacity of the foundation at the base of the barrier shall be not less than the barrier capacity in order to prevent failure of the foundations.

In accordance with the requirements of AS 5100.2:2017, the bridge approach barrier, connections and supporting system shall be designed as a progressive strength system to limit the damage to the supporting system and potential reconstruction requirements that may arise from a collision.

7. Bridge Support Protection Barrier

As defined by AS 5100.2:2017, support elements are piers, columns, abutments and walls that provide vertical supports to overhead structures. They also include support elements for developments adjacent to or over a roadway.

The supports of bridges as defined above are required to be structurally protected from vehicle collisions. They are considered roadside hazards and therefore need to have safety treatments to protect road users.

It is specified in Section 15 of AS 5100.1:2017 that the relevant jurisdictions need to assess the risk of a vehicle impacting bridge supports, bridge superstructures and bridge elements above the bridge deck, and determine the level of protection and the associated appropriate performance levels of the barrier systems. A detailed procedure for risk assessment and determination of the protection level, however, is not available. This section provides a design guideline for bridge support protection barriers considering the following:

- design requirements
- determination of level of protection
- barrier layout and design.

7.1 Current design and specification requirements

7.1.1 Current AS 5100 Requirements

AS 5100.1:2017 specifies that:

- For road traffic collisions, the relevant authority shall determine the minimum clearance of a pier/column from the roadway beyond which road traffic barrier protection will not be required.
- For pedestrian bridges, supports shall be either located to avoid collisions from road traffic or protected from such collisions.
- For railway traffic collisions, bridges over railways shall have a clear span between abutments, unless approved otherwise by relevant authorities. For road or pedestrian bridges over rail, where the nearest face of the pier is within 10 m from the centreline of existing or proposed tracks, the pier shall be a frangible pier in accordance with Section 15.3.3 AS 5100.2:2017. If a frangible pier is not practicable, the pier shall be protected from head-on impact from derailed trains by deflection walls (Section 15.3.4 AS 5100.2:2017) unless approved otherwise by relevant authorities.

Section 11 of AS 5100.2:2017 specifies the design collision loads and design requirements for protection of bridge supports from collisions of roadway and railway traffic. For collisions from road traffic, bridge supports shall be designed for a minimum equivalent static load of 2,700 kN applied in any direction in a horizontal plane of the road centreline and at 1.2 m above the ground level, if they are not located behind appropriate protective traffic barriers.

For collisions from rail traffic, AS 5100.1:2017 and AS 5100.2:2017 provide guidance with the intent of avoiding the collapse of structures over rail due to impact from derailed trains and reduce the severity of impacts with structures to reduce the probability of injury to occupants of derailed trains. Supporting elements located within 10 m of the centreline of the railway track shall be designed to resist a minimum collision load of 4,000 kN parallel to the rails and 1,500 kN normal to the rails. These loads are to be applied horizontally at 2 m above rail level distributed over a length of 2.0 m and 0.5 m. Where supporting elements are located between 10 m to 20 m from the centreline of the railway track, a risk analysis shall be carried out and approved by the relevant rail authority, which shall determine the required level of protection. If the level of redundancy does not meet the redundancy requirements of AS 5100.1, the support elements shall be designed to resist a minimum collision load of 1,500 kN at any angle in the horizontal plane directed towards the support from the adjacent track centreline, applied at 2 m above the ground level adjacent to the support.

For individual column shafts, the load should be considered as a point load. Columns that are 760 mm in diameter and smaller are the most vulnerable to impacts. The load may be considered to be a point load or may be distributed over an area deemed suitable for the size of the structure and the anticipated impacting vehicle, but not greater than 1,500 mm wide by 600 mm high. These dimensions were determined by considering the size of a truck frame (AASHTO 2020).

7.2 Design Procedure

The following design steps are suggested:

- Assess the site conditions to determine the level of protection, including the selection of an appropriate barrier performance level.
- Determine the barrier layout, including the working width, barrier LON and lateral offset of the barrier.
- Determine structural solutions, including barrier type, height consideration and foundations.

These design steps are presented in more detail in the following sections.

7.3 Determination of Level of Protection

Austrroads (2024) suggests that barriers shielding gantries and bridge piers are TL-5 concrete barriers as a default. Barriers that are 1,370 mm high and have an F-shape profile or single slope are preferred.

7.3.1 Working Width

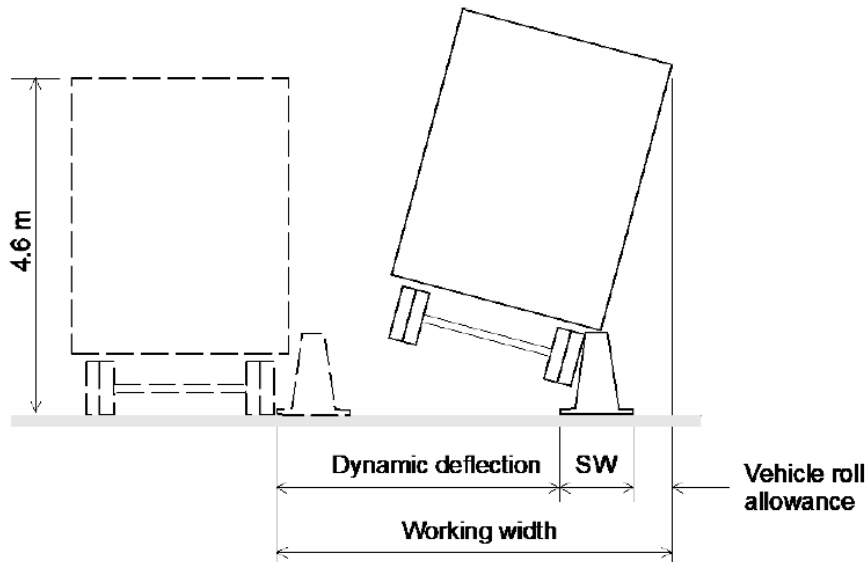
In design, the working width is the required clearance between the face of a road safety barrier and a hazard (AS 3845.1:2015). The working width is measured as the distance between the traffic face of the barrier before impact and the maximum lateral position of any major part of the system or vehicle during and after the impact.

The working width is recorded during full-scale crash testing and is summed from the following 3 sub-elements (Austrroads 2024):

- Dynamic deflection – defined as ‘the largest transverse deflection of any part of a road safety barrier system recorded during a full-scale crash test’. Note the dynamic deflection differs from the permanent deflection which is the deflection of the barrier after the impact.
- System width – the width of the system during or after impact and including any deformation. The system width is zero if the system is crushed or embedded into the vehicle, as occurs in some impacts with wire rope barriers.
- Vehicle roll allowance – is the lateral distance a vehicle protrudes beyond the deflected barrier. If the vehicle does not protrude beyond the deflected barrier, then the vehicle roll allowance is zero.

Figure 7.1 provides a graphical representation of the different components of working width. Minimum working widths for permanent concrete barriers are given in Table 7.1. A factor of safety should be used when these values are applied in a design (refer Section 6.13.1 of the Austroads *Guide to Road Design* Part 6 [Austroads 2024]).

Figure 7.1: Working width measurements showing dynamic deflection, system width (SW) and vehicle roll allowance



SW = System width

Source: Austroads (2024).

Table 7.1: Minimum working widths for permanent concrete barriers; vehicle maximum height of 4.6 m

Test level	Test	Barrier height	Working width
TL-3	3-11	915 mm	0.5 m*
TL-4	4-12	915 mm	2.3 m
		1,070 mm	2.2 m
TL-5	5-12	1,070 mm	2.4 m
		1,370 mm	1.5 m

*The working widths for TL-3 barriers are much smaller than those for other tests because of the lighter and lower test vehicle.

Source: Austroads (2024).

Austroads (2024) requires evaluations of critical infrastructure within the working width of a barrier to be accompanied with a design exception report that is approved by the relevant jurisdiction, and which includes:

- justification that all efforts have been made to minimise the working width, including providing a taller or less flexible system
- justification that a continuous hazard is smooth and free from snag points
- justification that debris from infrastructure dislodged following impact will not result in unacceptable safety risk to general traffic
- justification that if any snag points exist, they are outside of the working width
- a maintenance protocol for repair or reinstatement of infrastructure in the event it is damaged by heavy vehicles.

7.3.2 Levels of protection

When assessing the need for protection of bridge supports for collision from road traffic, risk assessment and determination of protective measures are carried out. Clauses 15.1 and 15.2 of AS 5100.1:2017 require that the relevant authorities undertake an assessment, which may result in one of the levels of protection discussed below.

No protection

Bridge supports located outside of the road reservation at a distance considered suitable by the transport agencies do not need to be designed for collision from road traffic. New bridge designs should take this into consideration.

Flexible and semi-rigid barriers

Flexible or semi-rigid barriers may be provided to protect occupants of errant vehicles in the vicinity of bridge supports where the bridge supports are located within the road reservation and there is sufficient space to accommodate the working width, including the barrier's dynamic deflection (Section 7.3.1) and vehicle roll allowance (Section 7.3.1). These barriers shall be designed in accordance with Austroads (2024), AS/NZS 3845.1:2015 and AS/NZS 3845.2:2017.

Under these circumstances, bridge supports shall be designed for the collision load specified in Section 7.1.1.

Rigid barriers

Where rigid barriers are provided, the barriers shall be structurally and geometrically designed in accordance with AS 5100:2017 for an appropriate performance level determined in Section 7.3.4. The barriers shall be crashworthy, ground-mounted and structurally independent.

The following should be noted when considering the adoption of a high containment rigid barrier:

- Road safety barriers suitable for all trucks are rarely used and are expensive.
- A heavy vehicle will not be contained by a Test Level 3 barrier.
- A car may be extensively damaged by impact with a barrier designed for trucks if it is not appropriately designed.

Except for barriers associated with bridges and other situations where the consequences of vehicles leaving the road are extreme, road safety barriers are not normally designed to contain van or tanker type semi-trailers (of Medium and Special performance levels). This design limitation has been practised primarily because of the relatively low volumes of these vehicles on many roads and the high cost of providing barriers to contain them.

A risk assessment as presented in Section 7.3.3 may be carried out to determine if rigid barriers should be provided.

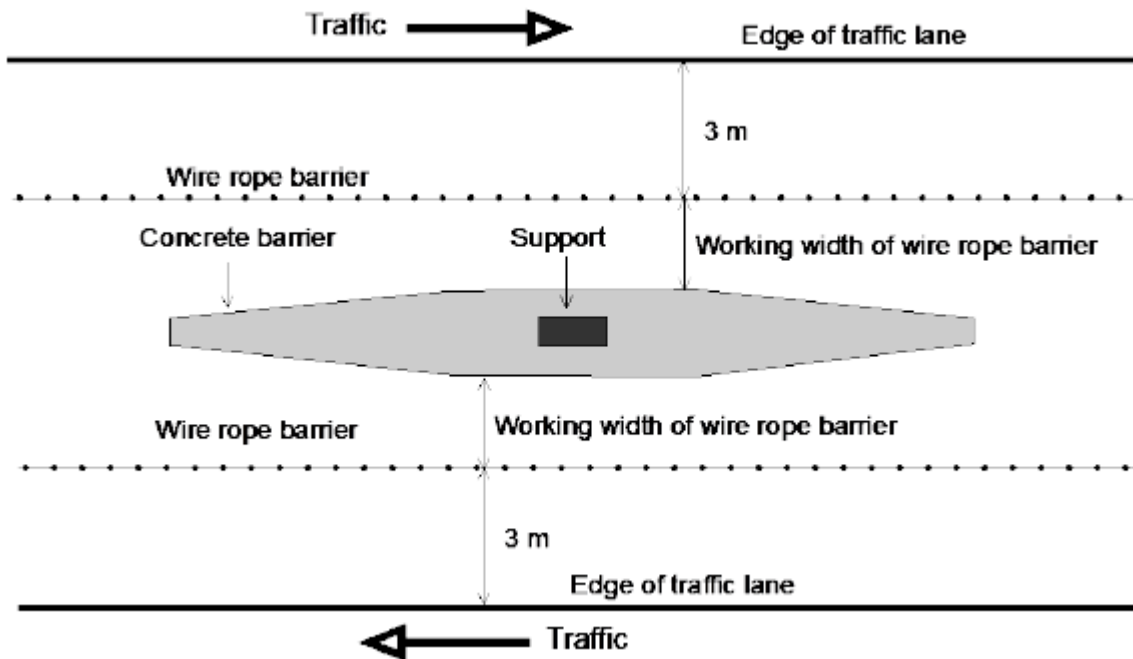
Where a risk assessment indicates that the run-off-road risk associated with heavy vehicles is particularly high, a Medium or Special performance level may be considered. If available, local information on truck encroachment frequency should also be considered.

At the Bridge Task Force meeting in December 2012, an agreement was reached by all jurisdictions on the principle that the bridge piers should be designed for collision from road traffic, regardless of the presence of roadside barriers.

Two-stage shielding

Two-stage shielding is a design solution for bridge piers in wider medians provided in Austroads (2024). A two-stage barrier system incorporates a concrete barrier and a more flexible one in front (Figure 7.2). A two-stage protection system can provide a barrier system to redirect passenger vehicles and a high containment barrier for the mitigation of additional risks from heavier vehicles. Flexible barrier systems are preferred to reduce the impact severity on passenger vehicle occupants. Additional information regarding a two-stage approach is provided in Appendix D.2.2 and AGRD Part 6 (Austroads 2024).

Figure 7.2: Example of two-stage protection



Source: Austroads (2024).

7.3.3 Risk assessment

Austroads (2024) requires a risk assessment to justify whether the use of a TL-4 or TL-5 containment level is appropriate at these sites or whether a higher containment level is warranted. A TL-6 containment level, to redirect tanker type articulated vehicles, could be used at higher risk sites, alternatively the containment level for a barrier could be defined by a performance level described in AS 5100.1. A site-specific risk assessment needs to be undertaken as per Austroads (2024).

For informational purposes, AASHTO (2020) provides a formula to evaluate the annual frequency of bridge collapse from impact with heavy vehicles, subject to approval by the relevant jurisdiction.

7.3.4 Selection of performance level

Where road safety barriers are provided, the barrier performance level and type shall be selected in accordance with Austroads (2024), based on the specific requirements of the site such as the design vehicle and speed environment.

Where a rigid barrier is required to either protect road users or bridge supports, the type and performance level of the barrier must be determined based on the traffic data and the site conditions. The following scenarios are suggested:

- Roads with low volumes of commercial vehicles (less than 15% AADT or 1500 commercial vehicles per day per carriageway): May consider a regular (TL4) performance level barrier (below the default TL-5), however this should be justified via the risk assessment in Section 7.3.3.
- Roads with medium to high volumes of commercial vehicles (from 15% AADT to 30% AADT or up to 3000 commercial vehicles per day per carriageway): Medium (TL5) performance level barrier should be provided.
- Roads with high volumes of commercial vehicles (greater than or equal to 30% AADT or 3000 commercial vehicles per day per carriageway); roads with special site-specific conditions as specified by relevant road jurisdiction: Special (TL6+) performance level barrier should be provided.
- Site-specific, unusual conditions and critical locations as specified by the relevant road jurisdiction: Special (TL6+) performance level barrier should be provided.

The Regular (TL4) performance level is the minimum performance level to be selected where rigid barriers are required.

8. Retrofit of Barriers

8.1 Reasons for retrofitting bridge barriers

This section provides guidance on the retrofitting of traffic barriers on existing bridges. The design of replacement or new traffic barriers for existing bridges is generally affected by the higher barrier design loads specified in current design standards compared to when the bridge was built. In NSW, for example, most bridges built before 2005 were designed using loads significantly less than those specified in the first iteration of AS 5100, published in 2004.

The following situations are generally considered for barrier retrofitting:

- where on-going vehicle collision costs and safety risks are excessive and a retrofit will reduce the collision frequency, severity of damage or cost of repair
- where the bridge is being widened or rehabilitated, the barrier shall be brought up to current standards
- where improvements are made to any bridge or its approach roadway, the bridge barriers shall be evaluated for conformance to current standards.

Other reasons that may be considered include:

- the need for separating pedestrian or cycle traffic from vehicle traffic for safety reasons due to an increase in the traffic volume
- handrail systems that do not have sufficient strength to comply with the AS 5100.2:2017 load requirements
- kerbs that do not have sufficient strength to comply with the AS 5100:2017 requirements
- kerbs that do not have sufficient height for vehicle re-direction
- kerbs that have a 'flat' face slope allowing easy vehicle mounting.

Bridge widening, which require retrofitting of barriers, is typically considered in the following situations:

- widening an inadequate roadway width on an existing bridge
- adding lanes to a highway segment to increase the traffic carrying capacity
- adding an auxiliary lane across the bridge.

8.2 Barrier retrofitting design methodology

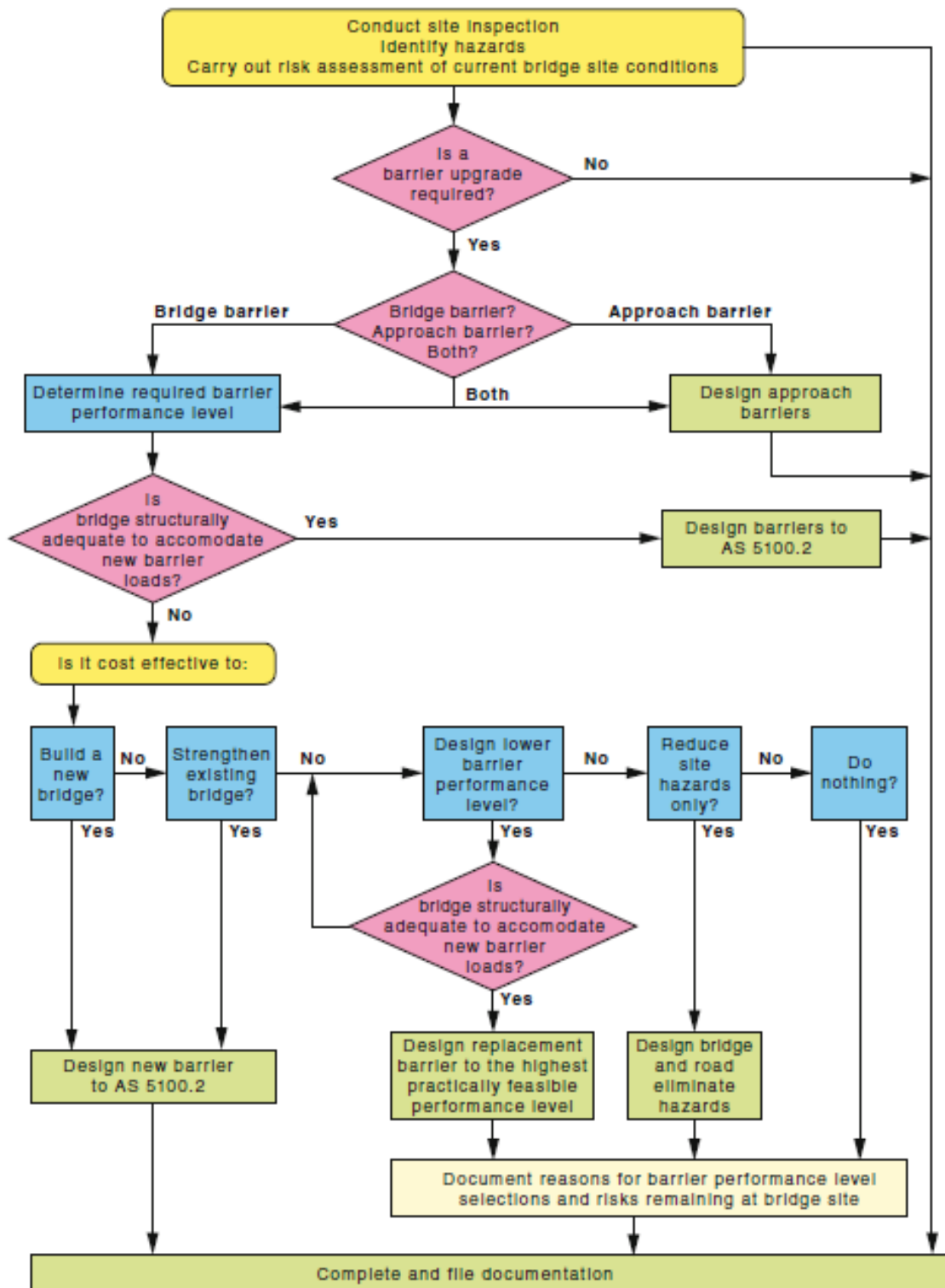
AS 5100.8, which was added to the AS 5100 series in 2017, identifies the process for designing a replacement traffic barrier for bridges, as shown in Figure 8.1. The following points outline the considerations for the replacements traffic barrier:

- A risk assessment should be undertaken in accordance with AS 5100.1:2017 to identify the required performance level.
- The barrier should be consistent along the road length being considered.
- If the bridge cannot be economically strengthened based on the performance level identified, the highest barrier performance level within the economic constraints should be adopted.
- Where the performance level is less than required from the risk assessment, alternative strategies should be considered, including reduced speed limits and advanced warning signs.

MASH testing performance is an internationally accepted standard which could be considered for use instead of the performance specifications in AS 5100.

For more information on retrofit of barriers refer to Appendix B.

Figure 8.1: Design process for replacement bridge barrier



Source: AS 5100.8:2017.

8.2.1 Design procedure

The barrier retrofitting design procedure will involve the following four-staged process:

1. Determine the appropriate barrier performance level for the site using the procedure in Section 3.
2. Evaluate the existing bridge structure to determine the strength available and the capability of the structure in accommodating the design loads associated with the appropriate barrier performance level without superstructure strengthening using Appendix B.
3. If the appropriate barrier performance level is higher than what can be accommodated by the existing structure, then determine the optimum achievable performance considering superstructure strengthening options.
4. Design the barrier retrofitting which may be a new barrier design or a barrier upgrading design and may include the retrofitting of the existing structure.

The barrier retrofitting design procedure in Figure 8.1 is suggested. Detailed steps are provided in the following sections.

8.2.2 Determine appropriate barrier performance level

The appropriate barrier performance level for the site is the performance level required by the site-specific condition and traffic data as for a new bridge construction. The selection of this barrier performance level is not constrained by the structural condition of the existing bridge.

A site-specific risk analysis shall be carried out to determine the appropriate barrier performance level for the bridge site as per Section 3.

Where the performance level of a replacement or upgraded barrier meets the geometric requirements of a performance level in AS 5100.2 but does not achieve the strength requirement, its performance level shall be indicated, as a percentage for example, '60% medium performance level' (AS 5100.8:2017).

8.2.3 Rationalisation of barrier performance level

From the results obtained by an evaluation presented in Appendix B, in many situations it may not be possible to upgrade an existing bridge to the appropriate barrier performance level. Reasons for this may include:

- insufficient capacity of the existing deck
- logistics of the demolition and/or the retrofit process
- incompatibility of the appropriate barrier and the existing bridge
- high cost of retrofit (whole-of-life-cycle cost)
- remaining life of the bridge.

Where it is not viable to build a new bridge or strengthen to the appropriate level, a lower performance level may be justified by using site-specific risk evaluation and benefit cost analysis. Section 8.3 provides a suggested process to determine if a lower performance level is appropriate for the particular site. A Safe System approach should also be taken into consideration which should consider speed limit reductions to reduce the kinetic energy involved in any potential impact event.

8.2.4 Design methodology

The following design principles are suggested:

- The yield line failure analysis method adopted from AASHTO (2020) should be used.

- The retrofitted barrier system should allow for progressive collapse i.e. the failure of the barrier system should occur in the following order: barrier, anchorage then deck slab.
- The retrofit design should match the structural components of the existing structure, including splice locations, as far as practical.

8.3 Determination of appropriate barrier performance level

Where it is not viable to install the appropriate performance level barrier for the site, as determined in Appendix B, the procedure described in the following sections is suggested to determine the most suitable barrier retrofit design solution.

8.3.1 Site-specific risk assessment

The following typical hazards of existing bridge barriers should be assessed to provide additional information for the determination of the most suitable barrier retrofitting design solution (Roads and Maritime Services 2018):

- unprotected rigid barrier (e.g. barrier ending on an end post that does not have any guardrail attached on the approach side)
- elements that may spear vehicles
- discontinuities in horizontal rails
- protruding kerbs
- inadequate barrier height
- barrier too brittle/fragile (e.g. lightly reinforced concrete rails, timber barriers, etc.)
- barrier too flexible (e.g. chain mesh)
- inadequate guardrail connection
- inadequate guardrail anchorage
- approach barrier too flexible
- insufficient stiffness transition
- high embankments
- lack of containment.

8.3.2 Benefit-cost analysis

A benefit-cost analysis can be carried out to select the most cost-effective option for the site.

The following options may be considered:

1. Do nothing.
2. Strengthen the existing bridge to accommodate new barrier loads.
3. Reduce the required barrier performance level.
4. Reduce site hazards including speed reduction.
5. Build a new bridge.

It is possible to combine options 2 and 3, or 2 and 4, above.

The following should be taken into consideration (Roads and Traffic Authority 2024):

- If the bridge cannot be economically strengthened to install a traffic barrier to the barrier performance level specified in Section 3, then the highest barrier performance level achievable within the economic constraints applicable to the site shall be adopted.
- If none of the barrier performance levels specified in Section 3 can be provided, then, as a minimum, the hazards identified in Section 8.3.1 or as otherwise determined shall be minimised.
- Where the proposed barrier performance level is less than that required from the risk assessment, then alternative amelioration strategies must be considered.
- The resulting bridge traffic barrier design solution must take into account any adverse impact on vulnerable road users such as cyclists or pedestrians.
- Barrier designs must not introduce or allow the continued existence of the barrier hazards identified.
- The information gathering, risk assessment, design, decision-making and approval as well as as-built drawings shall be recorded.

8.3.3 Additional considerations

The following additional situations should be taken into account when selecting a barrier retrofitting solution (Queensland Department of Transport and Main Roads 2024).

Retrofitting of bridge balustrades

Historically, balustrades may have been installed between the road and footpath to stop pedestrians and cyclists falling off the footpath onto the road where there is a large vertical drop to the road or where the footpath is narrow. Vehicle impact on balustrades poses a hazard to road users and pedestrians. Balustrades shall not be installed on the roadside of a footpath/bikeway on new works. A traffic barrier should be used in these applications.

On existing bridges with balustrades next to the road, the balustrade may be allowed to remain in place if the speed limit is 70 km/h or less and commercial vehicles are less than 5% of the traffic volume. In all other situations, balustrades shall be removed and replaced by conforming barriers.

Retrofitting barriers on widened bridges

AS 5100.7:2017 requires that where a bridge is being widened, or substantially rehabilitated, full barrier replacement shall be undertaken in accordance with AS 5100.1 and AS 5100.2. The following action shall be undertaken:

- For bridges with a repeated accident history of vehicle impacts or vehicles overturning, the barriers shall be replaced with barriers conforming to AS 5100:2017.
- Where AS 5100.1:2017 would currently require a performance level higher than Regular, the bridge must be modified to provide a higher level of protection. Design exemptions shall be documented and require acceptance by the relevant road jurisdiction.
- Where a 'regular' level barrier or less is required by AS(NZS) 5100, and the deck cannot support the current barrier design loads, a risk analysis for an existing bridge shall be undertaken in accordance with AS 5100.1 Clause 14.5.1. Design exemptions shall be documented and require acceptance by the relevant road jurisdiction.
- Some existing barriers (concrete post with water pipe rails or concrete posts with balustrade) have little strength.

Replacement of barrier damaged by vehicle impact, flood or due to bridge maintenance

The original barrier on the bridge would have been installed to the criteria current at the time of design. The following action shall be undertaken:

- The barrier should be replaced with a barrier conforming to AS(/NZS) 5100 if the existing deck has sufficient structural capacity to support the design load.
- Where AS(/NZS) 5100 specifies a performance level higher than 'regular', the deck must be modified to provide a higher level of protection. If impractical or uneconomic, application for a design relaxation shall be submitted to the relevant road jurisdiction.
- Where a 'regular' level barrier or less is required by AS(/NZS) 5100, and the deck cannot support the current barrier design loads, a risk analysis for an existing bridge shall be undertaken in accordance with AS 5100.1 Clause 14.5.1. If the bridge conforms to these criteria, a barrier of a lesser performance level may be installed. The minimum strength for replacement rails is 50% of 'low' performance level. If impractical or uneconomic, application for a design relaxation shall be submitted to the relevant road jurisdiction.

Upgrading of existing traffic barriers on bridges

There is no requirement to replace barriers automatically just because a new code has been released. The barrier on the original bridge would have been designed to the design criteria current at the time of design. Upgrading of existing barriers is uncommon. However, based on a risk assessment or adjacent accidents or near misses, it may be decided to upgrade a barrier. The following actions shall be undertaken:

- The barrier would be replaced with a barrier conforming to AS(/NZS) 5100 if the existing deck has sufficient structural capacity to support the design load.
- Where AS(/NZS) 5100 would currently require a performance level higher than 'regular', the bridge shall be modified to provide a higher level of protection. If impractical or uneconomic, application for a design relaxation shall be submitted to the relevant road jurisdiction.
- Where a 'regular' level barrier or less is required by AS(/NZS) 5100, and the deck cannot support the current barrier design loads, a risk analysis for an existing bridge shall be undertaken in accordance with AS 5100.1 Clause 14.5.1. The minimum strength for replacement rails is 50% of Low performance. If impractical or uneconomical, application for a design relaxation shall be submitted to the relevant road jurisdiction.

8.4 Barrier retrofit design procedure

As part of the planning for a bridge that will be widened or rehabilitated, the following aspects shall be considered in the selection of a retrofit barrier:

- characteristics of the bridge location
- design for upgrading or replacement of existing barrier
- design for retrofitting existing structure if required
- design of connections
- details of approach barriers.

The details of potential retrofit designs, such as the following, and their effect on selection of a retrofit barrier should be reviewed carefully:

- placement or spacing of new anchor bolts or dowels
- reinforcement anchorages
- approach guard fence post positioning

- shoulder width required by the new barrier.

8.4.1 Characteristics of the bridge location

Evaluate details of the location, such as the following, and consider their effect on selection of a retrofit barrier:

- bridge structure's height above lower terrain or waterway
- approach roadway width, alignment and grade
- position of adjacent streets and their average daily traffic volume
- bridge design speed, average daily traffic volume and percentage of heavy vehicles
- accident history on the bridge.

The effect of a full-strength retrofit on the shoulder width of the bridge should be evaluated to ensure that a reduction in effective shoulder width or in sight distances at adjacent intersections will not increase the accident rate. The following should also be considered:

- bridge width, alignment and grade
- type, aesthetics and strength of existing barrier
- bridge length and its potential for posting speed limits.

8.4.2 Retrofitting the existing structure

Retrofitting of the existing structure, if required, should be based on the results from a detailed structural evaluation as presented in Appendix B for the selected barrier performance level.

Possible bridge components needing to be retrofitted when retrofitting the barrier include cantilevered deck slabs and external longitudinal girders.

Strengthening options for the deck slab may include:

- externally bonded fibre reinforced polymer (FRP) lamination
- post-tensioning
- additional cross-section (cross beam, slab thickness)
- deck slab replacement or overlay.

An appropriate strengthening option should be selected based on site-specific structural conditions.

8.4.3 Design of connections

Upgrading existing barriers

Bolt connections are commonly used for the connection between new attachments with the components of the existing barriers or with the deck slab.

Barrier replacement

The connection of the new barrier with the existing slab shall be designed so that it is capable of carrying the impact loads. Typical types of connection include:

- Bolted connection: The bolts can be embedded in the existing concrete slab or can be an expansion type. This connection can be used for steel post and rail barriers.

- Cast-in-place connection: The steel reinforcement of the new barrier is welded to the existing steel bars of the slab. This connection type is used for concrete parapet barriers.

The barrier anchorage should be designed using a progressive strength system approach, such that the barrier and anchorage fail prior to the failure of the supporting system.

8.4.4 Retrofitting of bridge approach barriers

The bridge approach barrier will be considered for upgrade in 2 situations:

- where a bridge barrier is retrofitted and the roadside barrier is of a flexible type or lower performance level
- where a roadway guard fence is upgraded but the existing bridge barrier will remain in place, customised bridge barrier to guard fence transitions may be required.

Refer to Section 6 for guidance on bridge approach barrier design.

8.5 Bridge barrier upgrade exceptions

Rarely, but occasionally, an upgrade to a barrier on an existing structure could degrade rather than improve bridge safety. A possible design exception should be considered when planning a bridge-widening or rehabilitation project in which raising the barrier to meet current standards will reduce overall safety. Factors to consider include the following:

- design speed of traffic in the bridge location
- resistance to impact of the existing barrier
- whether the bridge ends are intersections protected by stop signs or traffic lights
- whether the geometry is straight into, along and out of the bridge
- whether traffic on the bridge is one-way or two-way
- accident history on the bridge, including damage to and repairs of the barrier
- risk of falling over the side of the bridge
- whether the bridge has a lower roadway
- whether a barrier upgrade will further narrow an already narrow lane width.

8.6 Special considerations for historic bridges

Federal law protects historically significant bridges and any rehabilitation or improvement projects on them require special attention. Bridges that are designated historically significant and that are listed or eligible to be listed in the National Heritage List introduce a special design challenge because the appearance of the bridge may be protected even though the historic barrier may not meet current standards.

The original barrier on a historic bridge is unlikely to meet current design requirements, particularly the structural capacity and barrier height requirements.

Options for upgrading the barrier on historic bridges usually include the following:

- Introduce an approved traffic barrier alongside the existing barrier, leaving the existing barrier in place. This is sometimes appropriate when a pedestrian walkway exists on or is planned for the bridge.
- Replace the existing barrier with an acceptable approved barrier.

- Remove the current barrier and incorporate it into a new acceptable barrier
This may be appropriate in rare instances where an existing barrier is especially decorative.
- Design a special barrier to match the appearance of the existing barrier.

9. Risk-Based Approach

Depending on the jurisdiction (refer to local guidance), undertaking a benefit cost analysis, in addition to the minimum specifications, is a possible approach to determine the most economic performance level. The performance level is selected from the highest level determined by minimum specifications and benefit cost analysis. However, this may not align with the Safe System approach (SSA). To ensure alignment with the SSA, the SSA assessment framework should be considered when choosing bridge barriers and considering the need to reduce barrier performance. In order to ensure that the SSA elements are being considered, the SSA assessment framework considers how well the elements of the project align with the SSA. Austroads provides a methodology for network, program and project risk assessments in the *Austroads Guide to Road Design Part 6* (Austroads 2024). The framework (Austroads 2016) assesses the major crash types and the contributing factors, which include:

- **Road user exposure:** This refers to the type of road user as well as the volume and duration on the road, which translates to the exposure for a potential crash. Key considerations for exposure include AADT, side-road traffic volumes, number of motorcycles, cyclists, and pedestrians crossing or walking along the road, length of the road, area and length of time.
- **Crash likelihood:** This looks at the factors which affect the probability of a crash occurring. This can include physical elements such as conflict points and offset to roadside hazards, as well as behavioural elements. Key considerations for crash likelihood include road user behaviour, road environment, level of intersection control, speed, sight distance, geometric alignment, driver guidance and warning as well as maintenance.
- **Crash severity:** This looks at the factors which affect the probability of a fatal or serious injury outcome should a crash occur, particularly the amount of kinetic energy likely transferred in the type of crash, such as impact speeds, angles and severity of roadside hazards.

Additional considerations for the Safe System approach components include the road user (e.g. are users likely to be fatigued), vehicle (e.g. large proportion of heavy vehicles) and post-crash care (e.g. are there access issues).

A quantitative benefit cost evaluation includes an assessment of the risk, which can be determined by evaluating the exposure likelihood and crash severity metrics as described above, and computation of the cost of crashes over the life of the structure, typically done on either a human capital approach or willingness to pay model (Department of Infrastructure, Transport, Regional Development and Communications 2022). This information can then be used together with installation, construction and maintenance costs to undertake benefit cost analysis.

Benefit cost comparisons can be made between barrier design options to inform the decision-making process. If the estimated benefit of a safer design option exceeds the cost of constructing and maintaining that design over a period, the safer design should be implemented subject to project funding constraints.

A benefit cost analysis should consider the period (project life) over which each alternative treatment provides a benefit. Since different treatments can have different project lives, both benefits and costs must be annualised so that direct comparisons between alternative treatments can be made. To reduce total costs to annualised costs, discount rates must be considered. An annualised benefit cost ratio (BCR) thus compares the expected savings to society (through reduced crash costs) to the costs (construction and maintenance) incurred to provide a specific treatment.

Additional information for benefit cost analysis is provided in Appendix C.

10. MASH Full-Scale Crash Testing

To provide greater context on the requirement in AS 5100:2017 to perform full-scale crash testing of bridge barriers in accordance with MASH (AASHTO 2016), full-scale crash testing was performed on a medium performance barrier. The purpose of this testing was to demonstrate compliance to AS 5100 and AS 3845.1:2015, assess the performance of a commonly used road traffic barrier in Australia and to provide a case study for how the MASH test protocol can be implemented in an Australian context. The test requirements and learnings are provided in this section, sourced from the full test report provided in Ryan and Ryder (2023).

10.1 Crash testing standards and guidelines

It is a requirement of AS 5100:2017 that the performance of all bridge traffic barrier types is demonstrated through full-scale crash tests (or be equivalent to an existing crash-tested system) following the MASH test protocol developed by the American Association of State Highway and Transportation Officials (AASHTO).

The MASH test protocol provides uniform guidelines for the crash testing of both permanent and temporary highway features for a range of performance levels. It assesses the safety feature on the following performance criteria:

- risk of injury to occupants of the impacting vehicle
- structural adequacy of the safety feature
- exposure to workers and pedestrians who may be behind the barrier or in the path of debris resulting from impact with a safety feature
- post-impact behaviour of the test vehicle.

MASH provides detailed requirements and guidance to ensure tests are performed consistently, the manual includes:

- test matrices and conditions
- test installation
- test vehicle specifications
- test evaluation criteria
- test documentation
- in-service performance evaluation.

The required MASH test scenarios for each barrier performance level are set out in AS 5100.1:2017 and shown in Table 3.1.

There is an extensive amount of work involved in planning, performing and reporting on the crash testing. This is especially true for barriers with regular performance level and higher as they require an additional crash test with an articulated truck between 10 and 36 tonnes. A high level of testing experience, quality control and expert supervision is required to successfully complete the tests. The steps involved in testing include:

- selection and design of the test barrier system including requirements for connection to bridge deck and length of test article
- identification of crash tests required for the barrier system

- design of foundations and/or a custom bridge deck that meet barrier performance criteria
- selection of a suitable testing location that meets the testing requirements
- Finite Element Method (FEM) modelling analyses to provide a preliminary estimate of barrier behaviour during the test, if accurate, can result in a more cost-effective test (optional)
- construction of the test article including barrier system, pavement, foundations and bridge deck
The test set up is required to be constructed in accordance with relevant quality control standards.
- procurement of test vehicles and installation of control systems (e.g. tow and cable guidance system, GPS control, etc.)
- installation of tri-axial accelerometers and single-rate gyro in test vehicles
- photographic and high speed and normal speed video documentation
- performance of 2 to 3 crash tests
- repair of the test pad and/or barrier structure between crash tests
- data processing and analysis
- verification of compliance according to the MASH standard
- reporting.

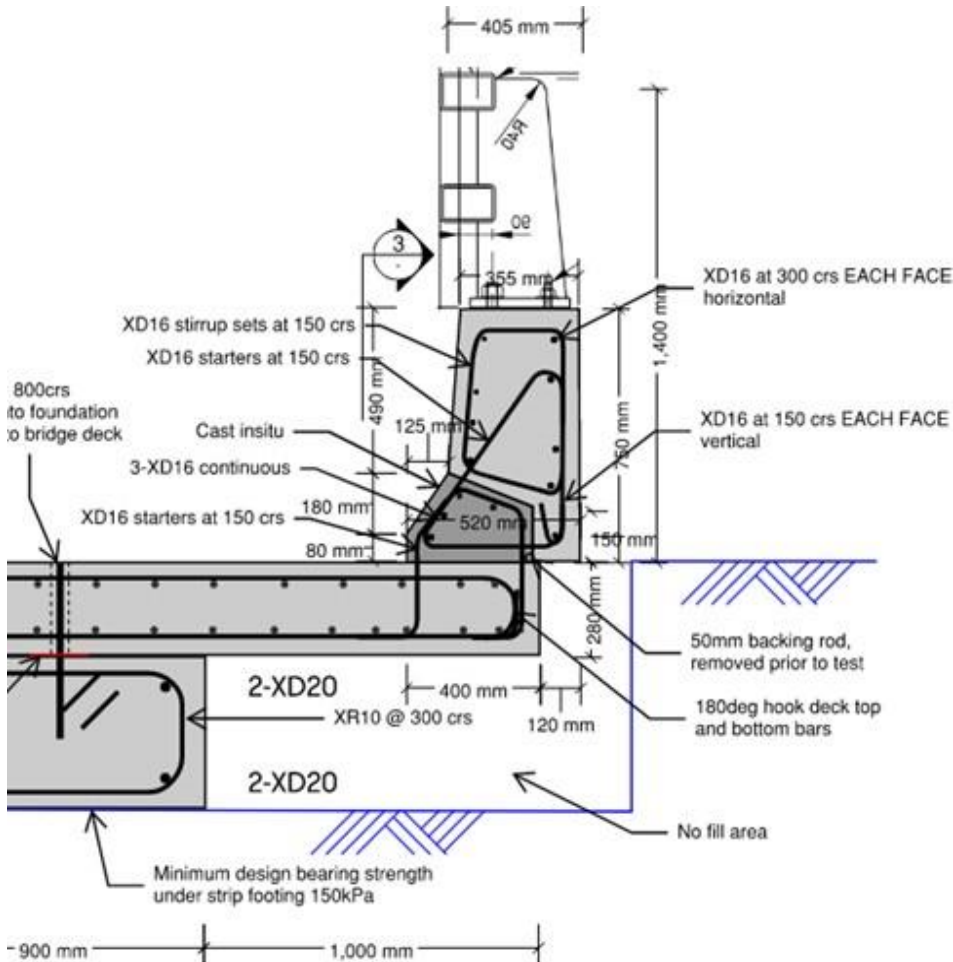
10.2 Compliance testing of the TfNSW medium performance traffic barrier type MAO

The full-scale crash testing was performed by Holmes Solutions (Ryan and Ryder 2023). Further details are available on the Austroads website at: <https://austroads.com.au/infrastructure/bridges/bridge-barrier-design>.

The barrier selected for full scale crash testing was a TfNSW MAO standard medium performance bridge barrier design. As per Table 3.1, a medium performance barrier is required to be tested against modified MASH test level 5 (TL5). TL5 requires 3 tests to be conducted: the first with a 1,100 kg car, the second with a 2,270 kg pick-up truck and the third with a 36,000 kg truck with articulated trailer.

The test article was designed and constructed as per TfNSW drawing B0505, as shown in Figure 10.1. The barrier was rigidly mounted to a cantilevered test bridge deck which was designed in accordance with the requirements for a 'medium' barrier in AS 5100 and constructed for the tests. The bridge deck consisted of 280 mm thick reinforced concrete and had a 1 m cantilever from the edge of the supporting foundation to simulate the overhang of a bridge deck.

Figure 10.1: Crash test medium performance barrier system arrangement



Source: Ryan and Ryder (2023).

The barrier length of need was designed to be 27 m which meets the minimum requirement of 23 m for rigid barriers in Section 3 of MASH. The top segment of the barriers was precast and stitched to the deck using concrete infill. The Rectangular Hollow Section (RHS) railing consisted of 2 longitudinal rails, at a height of 1,400 and 1,100 mm above ground level. An expansion joint with a 100 mm gap was incorporated into the bridge deck and barrier system, where this section was identified as the weakest point of the barrier system and was targeted through the critical impact point selected for each crash test. Testing was conducted in a controlled environment at the Ruapuna International Raceway in Christchurch, New Zealand.

10.2.1 Test results

Crash tests were successfully completed for all 3 required tests: 5-10, 5-11 and 5-12. The test barrier met the performance criteria for all 3 crash test cases. A summary is shown in Table 10.1.

Table 10.1: TL5 crash test cases

Test no.	Vehicle	Impact speed, km/h	Impact angle, deg	Evaluation criteria ⁽¹⁾	Structural adequacy	Occupant risk	Post-impact vehicular response
5-10	1100C	100	25	A,D,F,H,I	Pass	Pass	N/A ⁽²⁾
5-11	2270P	100	25	A,D,F,H,I	Pass	Pass	N/A ⁽²⁾
5-12	36000T	90 ⁽³⁾	15	A,D,G	Pass	Pass	N/A ⁽²⁾

Notes:

- 1 Evaluation criteria detailed in Section 5 of MASH.
- 2 The post-impact vehicular response evaluation criteria are only applicable for terminals, crash cushions, support structures, work-zone traffic control devices and breakaway utility poles.
- 3 AS 5100 has a modified impact speed requirement of 90 km/h instead of 80 km/h for test case 5-12.

Crash test 5-10

The 5-10 crash test involved a 1,100 kg car impacting the barrier at 100 km/h at an angle of 25°. A 2007 Nissan Tiida was used for the test. The impacts of the crash test on the barrier system and test vehicle are shown in Figure 10.2. There was no permanent deflection and it retained its original profile with no evidence of damage to the concrete barrier. The vehicle impact damaged the expansion joint fixings which were required to be replaced as well as minor scraping and denting to the steel railings.

The test vehicle remained upright during and after the collision, with roll and pitch angles being within acceptable ranges. The test vehicle experienced damage to the designated crumple zones but no penetration was recorded to the occupant compartment.

Crash test 5-11

The 5-11 crash test involved a 2,270 kg pick-up truck impacting the barrier at 100 km/h at an angle of 25°. A 2015 fourth generation Dodge Ram 1500 Quad Cab was used for the test. The impacts of the crash test on the barrier system and test vehicle are shown in Figure 10.3. There was no permanent deflection and it retained its original profile with no evidence of damage to the concrete barrier. The vehicle impact caused minor damage to the expansion joint fixings requiring them to be repaired as well as minor scraping and denting to the steel railings which also required repair.

The test vehicle remained upright during and after the collision with roll and pitch angles being within acceptable ranges. The test vehicle experienced damage to the designated crumple zones as well as cracking to the front right window and windshield however there was no penetration of the occupant compartment and the crash test dummy was contained within the vehicle.

Crash test 5-12

The 5-12 crash test involved a 36,000 kg articulated truck impacting the barrier at 90 km/h at an angle of 15°. A 2005 Kenworth T404S heavy truck and enclosed trailer was used for the crash test. The impacts of the crash test on the barrier system and test vehicle are shown in Figure 10.4. The barrier was moderately damaged from the vehicle impact with cracking around the rail anchors and deformation of the rail segments. The damaged concrete section and the steel rail portions required replacement. The expansion joint cover required minor repair. The profile of the barrier system maintained its shape and all barrier components remained attached to their original locations. The upper rail permanently deflected 120 mm and the concrete barrier did not permanently deflect.

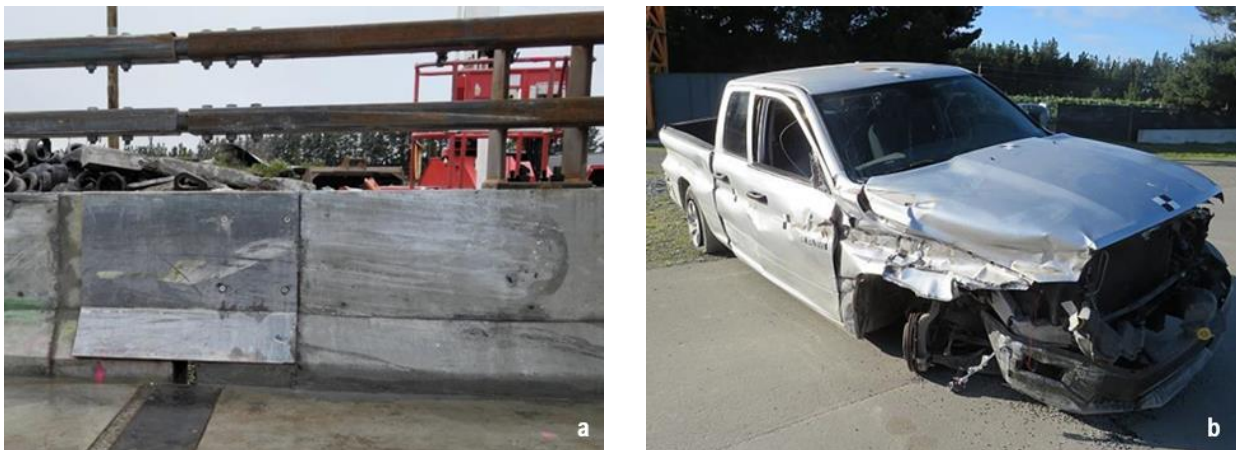
The test vehicle remained upright during and after the collision with roll and pitch angles being within acceptable ranges. The vehicle was moderately damaged with deformation to the front and undercarriage of the truck. There was no penetration of the occupant compartment.

Figure 10.2: Results of crash test 5-10 on (a) barrier system (b) test vehicle



Source: Ryan and Ryder (2023).

Figure 10.3: Results of crash test 5-11 on (a) barrier system (b) test vehicle



Source: Ryan and Ryder (2023).

Figure 10.4: Results of crash test 5-12 on (a) barrier system (b) test vehicle



Source: Ryan and Ryder (2023).

11. Finite Element Analysis

Computer simulation has been increasingly used to study the crashworthiness of vehicles as well as the behaviour of highway structures during collisions. The simulation models allow researchers to examine a variety of impact scenarios, test barrier configurations and test conditions that typically cannot be done by full-scale crash tests due to cost constraints. The simulation results can be used to assess deficiencies of existing road safety barriers and hardware and adjust these roadside safety features. Therefore, the design of roadside safety hardware can also be optimised based on the simulation results.

LS-DYNA, ABAQUS and Pam Crash are the typical programs utilising FEM used to develop simulation models. In the USA, computer simulation has been used as a tool to examine a range of barrier performance characteristics (as well as road safety hardware such as lighting poles, signage etc.) and provide the basis for improving the performance of the barrier before the actual crash testing (Williams et al. 2007).

Several comparisons have been made between computer simulation models and full-scale crash tests to validate the reliability of the computer models. Texas Transportation Institute (2020) used FE modelling to simulate crash testing for a transition barrier against MASH TL-3 evaluation criteria. LS-DYNA finite element modelling software was used for the simulation. The FE modelling led to a few design changes before full-scale crash testing was performed. Both the FE modelling and physical crash testing determined that the designed transition barrier satisfied MASH TL-3. Perneti, Scalera and Cibllis (2007) successfully carried out simulation tests for articulated trucks. They reported that the results obtained from the simulation models agreed well with that of full-scale crash tests. As a result, this model was validated to be used instead of full-scale crash tests. Drozda and Rotter (2017) also successfully used an FE model to accurately predict the results of a full-scale crash test for the scenario TB1 according to Eurocode EN 1317.

Various LS-DYNA compatible models are available for free download on established websites, such as those of the FHWA/NHTSA Crash Analysis Center and National Crash Analysis Center (NCAC). The computer models include every element involved in a crash: the vehicles, dummies with human biomechanical components and roadside hardware.

The rapid improvement in computer technology plays an important part in the development of the crash simulation models. Today, using low-cost parallel computing, NCAC, for example, can perform large-scale car-to-car simulations—complete with occupants and airbags—in just a few hours. The powerful, integrated workstations enable researchers to view and manipulate simulation data immediately.

Although there have been increased validations of computer models of barrier crash tests, computer simulation has not been accepted as an alternative for the full-scale crash tests, which is desirable for validation purposes.

In Australia, full-scale crash tests are required by AS 5100.1:2017 to be conducted to develop new bridge barrier designs. Computer simulation can potentially be used to produce assessments to corroborate the acceptance criteria specified in AS 5100.1 and especially to validate the barrier retrofitting designs.

For the modification and substitution of components in road safety barrier systems, AS/NZS 3845.1:2015 states that full-scale testing provides the most robust evidence of the effectiveness of a modification. It also identifies that simulation is becoming more common as expertise is developed worldwide, however If simulation is undertaken, then appropriate verification of the calibrated models needs to be undertaken. It is important that the model be calibrated so that the results compare favourably with known full-scale crash tests and other physical testing. A process for validating and verifying a simulation model against real world physical testing is identified in NCHRP Report W179 (Ray et al. 2010). It is important that the simulation model not be used to extrapolate significantly beyond the full-scale test results.

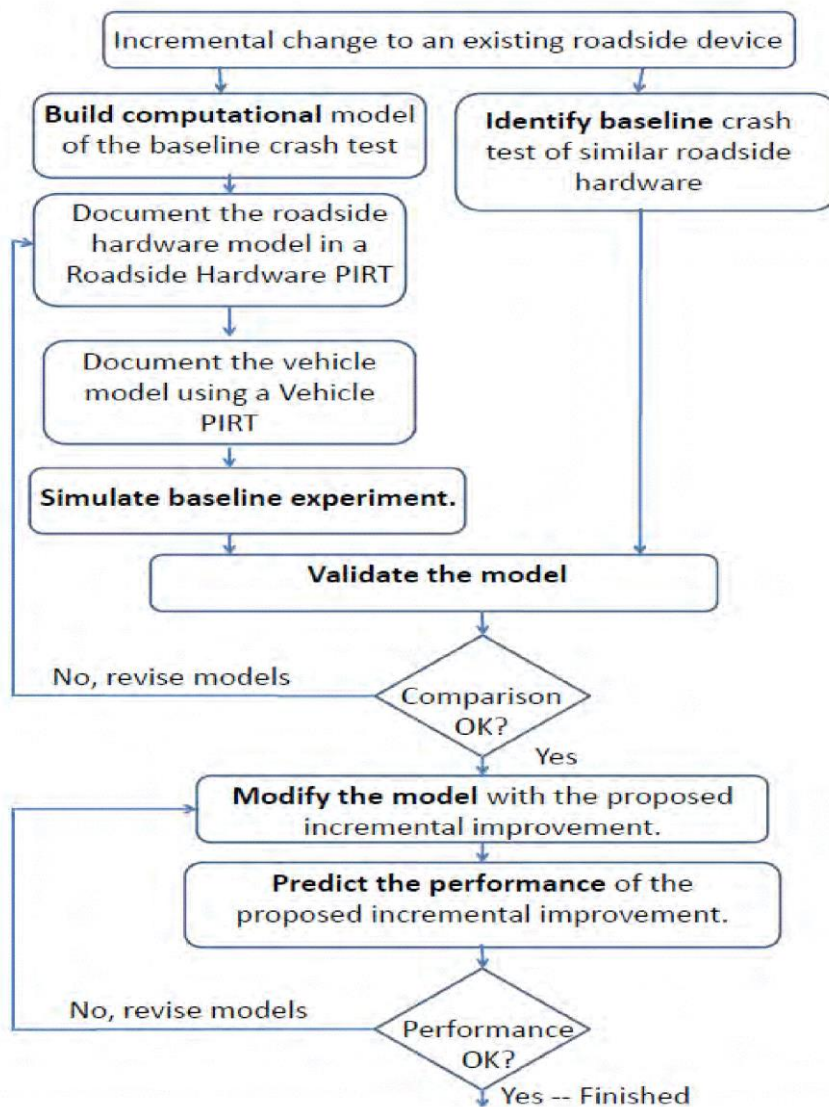
The process identified by Ray et al. (2010) is provided in Figure 11.1, the high-level steps covered include:

1. Identify experiment to use as baseline.
2. Build the computational model of the baseline experiment.
3. Use the model to simulate the baseline experiment.
4. Validate the model by comparing the simulation results with the physical test results.
5. Modify the model to represent incremental improvements.
6. Use the model to predict the performance of the incremental improvement.
7. Evaluate the performance of the incrementally modified device.

To assure the ASBAP that the model accurately represents the real-world physical testing, Austroads (2020) requires the following key documentation to allow assessment of simulated data:

8. A verification and validation report that documents the comparison between a full-scale crash test and the simulation inclusive of:
 - analysis solution verification table
 - phenomena importance ranking table
 - time history evaluation table
 - side by side photographs.
9. A simulation report documenting the results of the analysis of the untested design including hardware and vehicle data.

Figure 11.1: Barrier modelling process



Source: Ray et al. (2010).

11.1 FEA testing of the TfNSW medium performance traffic barrier type MAO

Finite element analyses (FEA) were conducted for the 3 MASH test cases in Section 10.2 for the TfNSW Medium Performance Traffic Barrier Type MAO prior to crash testing. Computer analyses were undertaken to achieve 2 outcomes:

- Provide confidence that the selected barrier design would perform as expected during the physical full-scale crash testing.
- Provide a baseline for further validation once the physical test had been run to enable future optimisation of the design or testing of similar designs.

The model was developed by GDTech, in consultation with Austroads, National Transport Research Organisation (NTRO) and Holmes Solutions, to ensure the model represented physical test conditions as closely as possible.

Simulated crash tests were successfully completed for all 3 required tests: 5-10, 5-11 and 5-12. The test barrier met the performance criteria for all crash test case simulations. A summary is shown in Table 11.1. This provided confidence that the physical full-scale crash tests would be successful. Renderings from the crash test simulations are provided in Figure 11.2, Figure 11.3 and Figure 11.4.

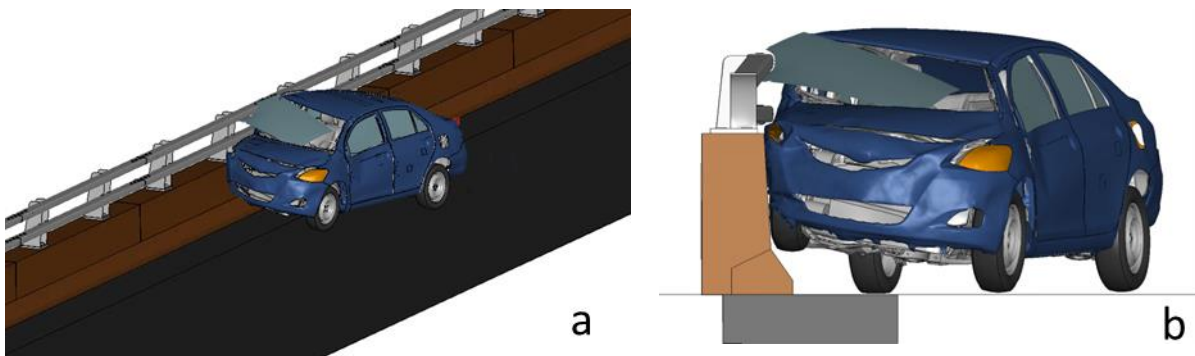
Table 11.1: Simulation TL5 crash test cases

Test no.	Vehicle	Impact speed, km/h	Impact angle, deg	Evaluation criteria ⁽¹⁾	Structural adequacy	Occupant risk	Post-impact vehicular response
5-10	1100C	100	25	A,D,F,H,I	Pass	Pass	N/A ⁽²⁾
5-11	2270P	100	25	A,D,F,H,I	Pass	Pass	N/A ⁽²⁾
5-12	36000T	90 ⁽³⁾	15	A,D,G	Pass	Pass	N/A ⁽²⁾

Notes:

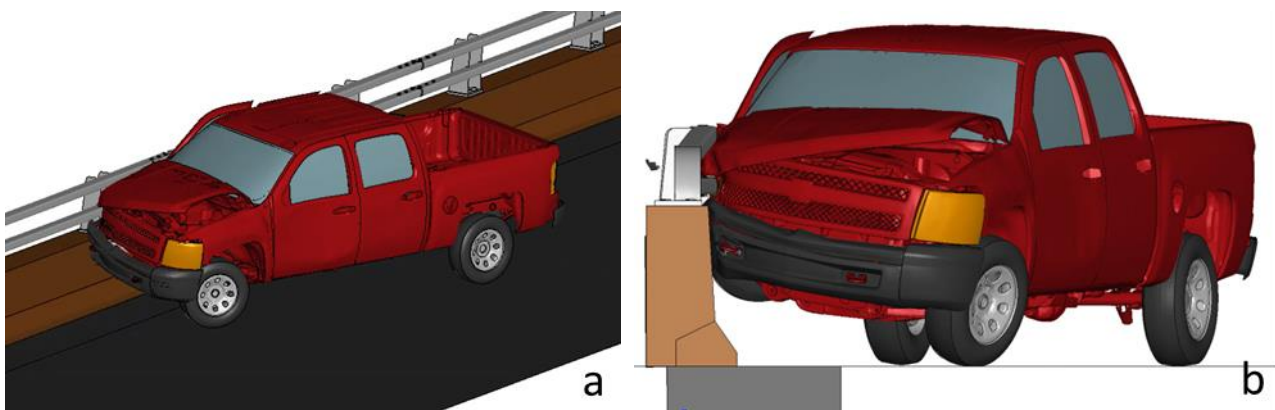
- ¹ Evaluation criteria detailed in Section 5 of MASH.
- ² The post-impact vehicular response evaluation criteria are only applicable for terminals, crash cushions, support structures, work-zone traffic control devices and breakaway utility poles.
- ³ AS 5100 has a modified impact speed requirement of 90 km/h instead of 80 km/h for test case 5-12.

Figure 11.2: Results of simulation crash test 5-10 from (a) isometric view (b) front view



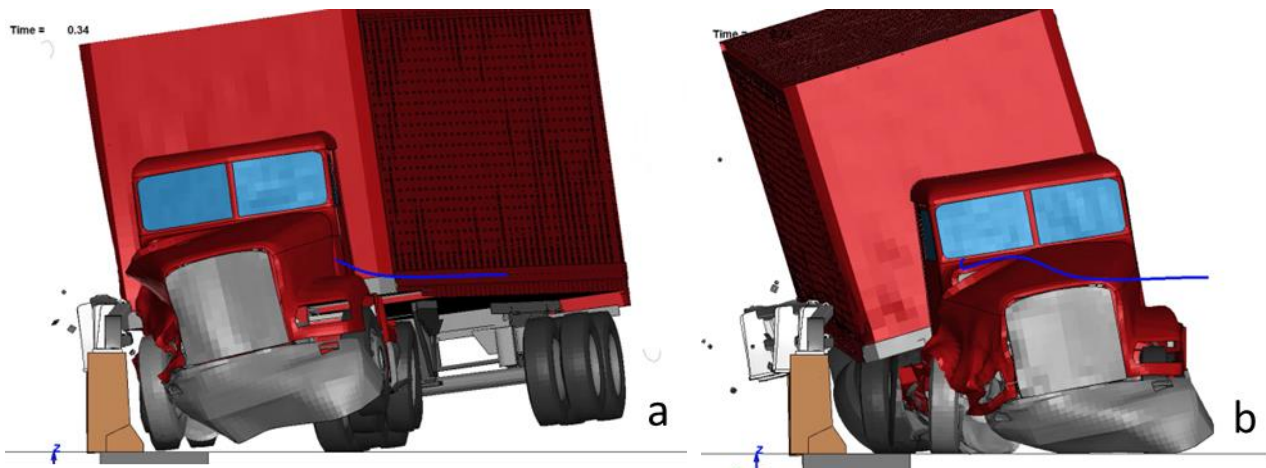
Source: Massaux (2023).

Figure 11.3: Results of simulation crash test 5-11 from (a) isometric view (b) front view



Source: Massaux (2023).

Figure 11.4: Results of simulation crash test 5-12 from (a) front view collision with cab (b) front view collision with trailer



Source: Massaux (2023).

11.1.1 Verification and validation

The verification and validation reports of the TfNSW MAO barrier model (Lucchese and Himpe 2024a, 2024b and 2024c) were produced according to the recommendations on NCHRP Report W179 (Ray et al. 2010) and provide a comparison between the full-scale crash test and the simulation including:

- analysis solution verification table
- phenomena importance ranking table
- time history evaluation table
- side by side photographs.

The complete reports for the tests 5-10, 5-11 and 5-12 are given in Lucchese and Himpe (2024a, 2024b and 2024c).

Analysis solution verification table

The analysis solution verification tables (Table 11.2, Table 11.3 and Table 11.4) are provided to ensure that the global energies and mass involved in the simulation remain within a reasonable range. This section aims to confirm that the numerical solution adheres to fundamental physical laws (e.g. conservation of energy, mass and momentum) and maintains numerical stability.

For the TfNSW Medium Performance Type MAO traffic barrier model, the analysis solution verification was successful in all the tests (TL5-10, TL5-11 and TL5-12), with one exception. This exception refers to the change in the mass of a fishplate on top of the anchorage plates connecting the railings to the concrete block. However, the margin to comply with the criterion is very low, and these parts, which behave normally in the simulation, do not play a major role.

Table 11.2: TL5-10 analysis solution verification table

Verification evaluation criteria	Change (%)	Pass?
Total energy of the analysis solution (i.e. kinetic, potential, contact, etc.) must not vary more than 10% from the beginning of the run to the end of the run.	0.36	Yes
Hourglass energy of the analysis solution at the end of the run is less than 5% of the total initial energy at the beginning of the run.	0.13	Yes
Hourglass energy of the analysis solution at the end of the run is less than 10% of the total internal energy at the end of the run.	0.39	Yes
The part/material with the highest amount of hourglass energy at any time during the run is less than 5% of the total initial energy at the beginning of the run.	0.07	Yes
Mass added to the total model is less than 5% of the total model mass at the beginning of the run.	0.00	Yes
The part/material with the most mass added had less than 10% of its initial mass added.	0.075 kg > 0.01 kg	No
The moving parts/materials in the model have less than 5% of mass added to the initial moving mass of the model.	0.00	Yes
There are no shooting nodes in the solution.	0	Yes
There are no solid elements with negative volumes.	0	Yes

Source: Lucchese and Himpe (2024a).

Table 11.3: TL5-11 analysis solution verification table

Verification evaluation criteria	Change (%)	Pass?
Total energy of the analysis solution (i.e. kinetic, potential, contact, etc.) must not vary more than 10% from the beginning of the run to the end of the run.	0.29	Yes
Hourglass energy of the analysis solution at the end of the run is less than 5% of the total initial energy at the beginning of the run.	0.07	Yes
Hourglass energy of the analysis solution at the end of the run is less than 10% of the total internal energy at the end of the run.	6.28	Yes
The part/material with the highest amount of hourglass energy at any time during the run is less than 5% of the total initial energy at the beginning of the run.	0.95	Yes
Mass added to the total model is less than 5% of the total model mass at the beginning of the run.	0.00	Yes
The part/material with the most mass added had less than 10% of its initial mass added.	0.075 kg > 0.01 kg	No
The moving parts/materials in the model have less than 5% of mass added to the initial moving mass of the model.	0.00	Yes
There are no shooting nodes in the solution.	0	Yes
There are no solid elements with negative volumes.	0	Yes

Source: Lucchese and Himpe (2024b).

Table 11.4: TL5-12 analysis solution verification table

Verification evaluation criteria	Change (%)	Pass?
Total energy of the analysis solution (i.e. kinetic, potential, contact, etc.) must not vary more than 10% from the beginning of the run to the end of the run.	0.38	Yes
Hourglass energy of the analysis solution at the end of the run is less than 5% of the total initial energy at the beginning of the run.	0.22	Yes
Hourglass energy of the analysis solution at the end of the run is less than 10% of the total internal energy at the end of the run.	0.32	Yes

Verification evaluation criteria	Change (%)	Pass?
The part/material with the highest amount of hourglass energy at any time during the run is less than 5% of the total initial energy at the beginning of the run.	0.12	Yes
Mass added to the total model is less than 5% of the total model mass at the beginning of the run.	0.00	Yes
The part/material with the most mass added had less than 10% of its initial mass added.	0.075 kg > 0.02 kg	No
The moving parts/materials in the model have less than 5% of mass added to the initial moving mass of the model.	0.00	Yes
There are no shooting nodes in the solution.	0	Yes
There are no solid elements with negative volumes.	0	Yes

Source: Lucchese and Himpe (2024c).

Phenomena importance ranking table

The phenomena importance ranking table provides evidence that the roadside hardware model is valid for its intended use. Table 11.5 is similar to the evaluation tables in AASHTO (2016) and the criteria are selected according to the test. For the TfNSW Medium Performance Type MAO traffic barrier model, the criteria were all passed, with exceptions for each test.

1. Test TL5-10: Evaluation Criteria A, D, F, H, I, M

For the TL5-10, the exception is regarding the maximum roll of the vehicle Criteria F – subcategory F2, which states that:

“Maximum roll of the vehicle: relative difference is less than 20 percent or absolute difference is less than 5 degrees” (Ray et al. 2010:E-11).

The results obtained were 44.1% and 7.5 degrees, respectively.

This discrepancy can be attributed to a particular experimental interaction between the car components and the system. Observations from the experimental crash frames indicate that the vehicle's behaviour upon impact with the system was not entirely replicated in the simulation. Specifically, the front right wheel was less damaged in the simulation compared to the actual crash, leading to differences in local interaction with the barrier and impact velocity. These differences significantly influenced certain experimental parameters, particularly the maximum roll of the vehicle observed after the impact with the system.

2. Test TL5-11: Evaluation Criteria A, D, F, L, M

For the TL5-11, the exception is in criteria L, subgroup L1, for the longitudinal occupant impact velocity that states:

“Occupant impact velocities: relative difference is less than 20 percent or absolute difference is less than 4 g's” (Ray et al. 2010:E-11).

The results obtained were 76% and 3.8 m/s, respectively.

This exception can be attributed to the specific experimental interaction between the car components and the system. Observations from the experimental crash frames reveal that the vehicle's behaviour when impacting with the system was not fully replicated in the simulation. Specifically, the front right wheel was not as damaged in the simulation as it was in the reality, resulting in different local interactions with the barrier and variations in impact velocity. This clearly influenced some of the experimental parameters obtained, in particular the longitudinal occupant impact velocity. Despite this exception, the other parameters from the simulation met the criteria, notably the theoretical head impact velocity (THIV) and the longitudinal occupant risk assessment (ORA), which are closely related to the longitudinal occupant impact velocity.

3. Test TL5-12: Evaluation Criteria A, D, G, M

For the TL5-12, there are 2 exceptions regarding the length of vehicle-barrier contact and the maximum roll of the vehicle (A and G).

A3 states:

“Length of vehicle-barrier contact: relative difference is less than 20 percent or absolute difference is less than 2 m.”

Values found were 48.3% and 5.6 m, respectively.

G2 states:

“Maximum roll of the vehicle: relative difference is less than 20 percent or absolute difference is less than 5 degrees.”

Values found were 50% and 13 degrees.

These exceptions can be attributed to the specific experimental interactions between the truck components and the system. As observed from the experimental crash frames, the behaviour of the truck upon impact with the system was not totally replicated in the simulation. The cab was more redirected in the simulation compared to reality, resulting in different local interactions with the barrier and variations in impact velocity. Additionally, the behaviour of the front axle was critical and appeared to differ in the simulation. These differences influenced certain experimental parameters, particularly the length of vehicle-barrier contact and the maximum roll of the vehicle. As the cab loses contact with the system, the truck trailer impacts the system with the rear and gets redirected along with the cab, preventing it from sliding along the system as observed in the actual crash test.

Table 11.5: Evaluation criteria test applicability table

Evaluation factors	Evaluation criteria		Applicable tests
Structural adequacy	A	Test article should contain and redirect the vehicle; the vehicle should not penetrate, under-ride, or override the installation although controlled lateral deflection of the test article is acceptable.	10, 11, 12, 20, 21, 22, 35, 36, 37, 38
	B	The test article should readily activate in a predictable manner by breaking away, fracturing or yielding.	60, 61, 70, 71, 80, 81
	C	Acceptable test article performance may be by redirection, controlled penetration or controlled stopping of the vehicle.	30, 31, 32, 33, 34, 39, 40, 41, 42, 43, 44, 50, 51, 52, 53
Occupant risk	D	Detached elements, fragments or other debris from the test article should not penetrate or show potential for penetrating the occupant compartment, or present an undue hazard to other traffic, pedestrians or personnel in a work zone.	All
	E	Detached elements, fragments or other debris from the test article, or vehicular damage should not block the driver's vision or otherwise cause the driver to lose control of the vehicle (answer Yes or No).	70, 71
	F	The vehicle should remain upright during and after the collision although moderate roll, pitching and yawing are acceptable.	All, except those listed in criterion G
	G	It is preferable, although not essential, that the vehicle remain upright during and after collision.	12, 22 (for test level 1–30, 31, 32, 33, 34, 35, 36, 37, 38, 39, 40, 41, 42, 43, 44)

Evaluation factors	Evaluation criteria			Applicable tests	
	H	Occupant impact velocities should satisfy the following:		10, 20, 30, 31, 32, 33, 34, 36, 40, 41, 42, 43, 50, 51, 52, 53, 80, 81	
		Occupant Impact Velocity Limits (m/s)			
		Component	Preferred		Maximum
		Longitudinal and Lateral	9		12
		Longitudinal	3	5	60, 61, 70, 71
	I	Occupant ride down accelerations should satisfy the following:		10, 20, 30, 31, 32, 33, 34, 36, 40, 41, 42, 43, 50, 51, 52, 53, 60, 61, 70, 71, 80, 81	
		Occupant Ride down Acceleration Limits (g's)			
		Component	Preferred		Maximum
		Longitudinal and Lateral	15	20	
Vehicle trajectory	L	The occupant impact velocity in the longitudinal direction should not exceed 40 ft/sec and the occupant ride-down acceleration in the longitudinal direction should not exceed 20 g's.		11, 21, 35, 37, 38, 39	
	M	The exit angle from the test article preferable should be less than 60% of test impact angle, measured at the time of vehicle loss of contact with test device.		10, 11, 12, 20, 21, 22, 35, 36, 37, 38, 39	
	N	Vehicle trajectory behind the test article is acceptable.		30, 31, 32, 33, 34, 39, 42, 43, 44, 60, 61, 70, 71, 80, 81	

Source: NCHRP Report W179 (Ray et al. 2010).

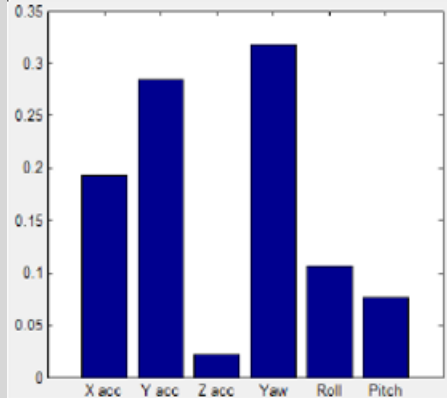
Time history evaluation table

The time history evaluation table has the objective to compare the time histories obtained in the model with those obtained from the physical experiments using appropriate metrics.

The computer program used to evaluate metrics in the verification and/or validation of numerical models in the roadside safety field is the Roadside Safety Simulation Validation Program (RSSVP) (Ray et al. 2010).

For all the tests of the TfNSW Medium Performance Type MAO traffic barrier model (TL5-10,11 and 12), the time history comparisons using multiple channels passed all the criteria (Table 11.6, Table 11.7 and Table 11.8).

Table 11.6: TL5-10 – Roadside safety validation metrics rating table – Time history comparisons (multiple channels)

Evaluation criteria (time interval [0 sec; 0.5 sec])																		
Channels (select which were used)																		
☒ X acceleration		☒ Y acceleration		☒ Z acceleration														
☒ Roll rate		☒ Pitch rate		☒ Yaw rate														
Multi-channel weights ☐ Area II method ☐ Inertial method		X channel: 0.19 Y channel: 0.28 Z channel: 0.02 Yaw rate channel: 0.32 Roll rate channel: 0.11 Pitch rate channel: 0.07 <table><caption>Multi-channel weights data</caption><thead><tr><th>Metric</th><th>Weight</th></tr></thead><tbody><tr><td>X acc</td><td>0.19</td></tr><tr><td>Y acc</td><td>0.28</td></tr><tr><td>Z acc</td><td>0.02</td></tr><tr><td>Yaw</td><td>0.32</td></tr><tr><td>Roll</td><td>0.11</td></tr><tr><td>Pitch</td><td>0.07</td></tr></tbody></table>			Metric	Weight	X acc	0.19	Y acc	0.28	Z acc	0.02	Yaw	0.32	Roll	0.11	Pitch	0.07
					Metric	Weight												
X acc	0.19																	
Y acc	0.28																	
Z acc	0.02																	
Yaw	0.32																	
Roll	0.11																	
Pitch	0.07																	
O	Sprague-Geer metrics Values less than or equal to 40 are acceptable.			M 13.5 P 24.3 Pass? Yes														
P	ANOVA metrics Both of the following criteria must be met: The mean residual error must be less than 5% of the peak acceleration $\overline{e} \leq 0.05 \times a_{Peak}$ The standard deviation of the residuals must be less than 35% of the peak acceleration $\sigma \leq 0.35 \times a_{Peak}$ Mean residual [%] -0.3 Standard deviation of residuals [%] 16.9 Pass? Yes																	
The Analysis Solution (check one) ☒ passes ☐ does NOT pass <u>all</u> the criteria in Table E-3.																		

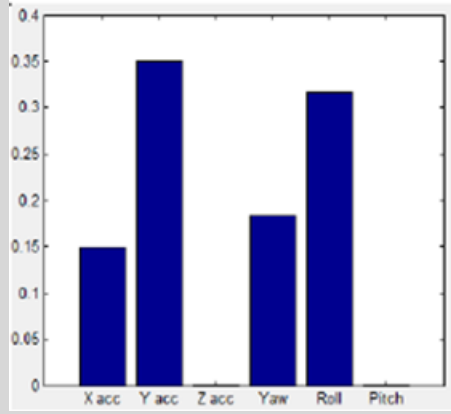
Source: Lucchese and Himpe (2024a).

Table 11.7: TL5-11 – Roadside safety validation metrics rating table – Time history comparisons (multiple channels)

Evaluation criteria (time interval [0 sec; 0.5 sec])																		
Channels (select which were used)																		
☒ X acceleration	☒ Y acceleration	☒ Z acceleration																
☒ Roll rate	☒ Pitch rate	☒ Yaw rate																
Multi-channel weights ☐ Area II method ☐ Inertial method	X channel: 0.15 Y channel: 0.22 Z channel: 0.03 Yaw rate channel: 0.37 Roll rate channel: 0.11 Pitch rate channel: 0.01	<table border="1"> <caption>Normalized Weights from Bar Chart</caption> <thead> <tr> <th>Channel</th> <th>Weight</th> </tr> </thead> <tbody> <tr> <td>X acc</td> <td>0.15</td> </tr> <tr> <td>Y acc</td> <td>0.22</td> </tr> <tr> <td>Z acc</td> <td>0.03</td> </tr> <tr> <td>Yaw</td> <td>0.37</td> </tr> <tr> <td>Roll</td> <td>0.11</td> </tr> <tr> <td>Pitch</td> <td>0.01</td> </tr> </tbody> </table>			Channel	Weight	X acc	0.15	Y acc	0.22	Z acc	0.03	Yaw	0.37	Roll	0.11	Pitch	0.01
Channel	Weight																	
X acc	0.15																	
Y acc	0.22																	
Z acc	0.03																	
Yaw	0.37																	
Roll	0.11																	
Pitch	0.01																	
O	Sprague-Geer metrics Values less than or equal to 40 are acceptable.	M	P	Pass?														
		22.8	29.0	Yes														
P	ANOVA metrics Both of the following criteria must be met: - The mean residual error must be less than 5% of the peak acceleration ($\bar{e} \leq 0.05 \times a_{Peak}$) - The standard deviation of the residuals must be less than 35% of the peak acceleration ($\sigma \leq 0.35 \times a_{Peak}$)	Mean residual [%]	Standard deviation of residuals [%]	Pass?														
		-1.6	27.7	Yes														
The analysis solution (check one) ☒ passes ☐ does NOT pass <u>all</u> the criteria in Table E-3.																		

Source: Lucchese and Himpe (2024b).

Table 11.8: TL5-12 – Roadside safety validation metrics rating table – Time history comparisons (multiple channels)

Evaluation criteria (time interval [0 sec; 1.0 sec])						
Channels (select which were used)						
☒ X acceleration		☒ Y acceleration		☒ Z acceleration		
☒ Roll rate		☒ Pitch rate		☒ Yaw rate		
Multi-channel weights ☐ Area II method ☐ Inertial method		X channel: 0.15 Y channel: 0.35 Z channel: 0.01 Yaw rate channel: 0.18 Roll rate channel: 0.32 Pitch rate channel: 0.01				
O	Sprague-Geer metrics Values less than or equal to 40 are acceptable.			M	P	Pass?
				28.3	39.9	Yes
P	ANOVA metrics Both of the following criteria must be met: - The mean residual error must be less than 5% of the peak acceleration ($\bar{e} \leq 0.05 \times a_{Peak}$) - The standard deviation of the residuals must be less than 35% of the peak acceleration ($\sigma \leq 0.35 \times a_{Peak}$)			Mean residual [%]	Standard deviation of residuals [%]	Pass?
				3.2	29.4	Yes
				The analysis solution (check one) ☒ passes ☐ does NOT pass <u>all</u> the criteria in Table E-3.		

Source: Lucchese and Himpe (2024c).

Side-by-side photographs

Qualitative comparisons can be made through side-by-side photographs of the model to establish if the model captures the basic phenomena involved in the impact. It also gives support to the quantitative comparisons and analysis of the previous tables.

Figure 11.5 to Figure 11.8 show the top view during the crash test for TL5-10, TL5-11 and TL5-12, respectively. Other views can be found in Lucchese and Himpe (2024a, 2024b and 2024c).

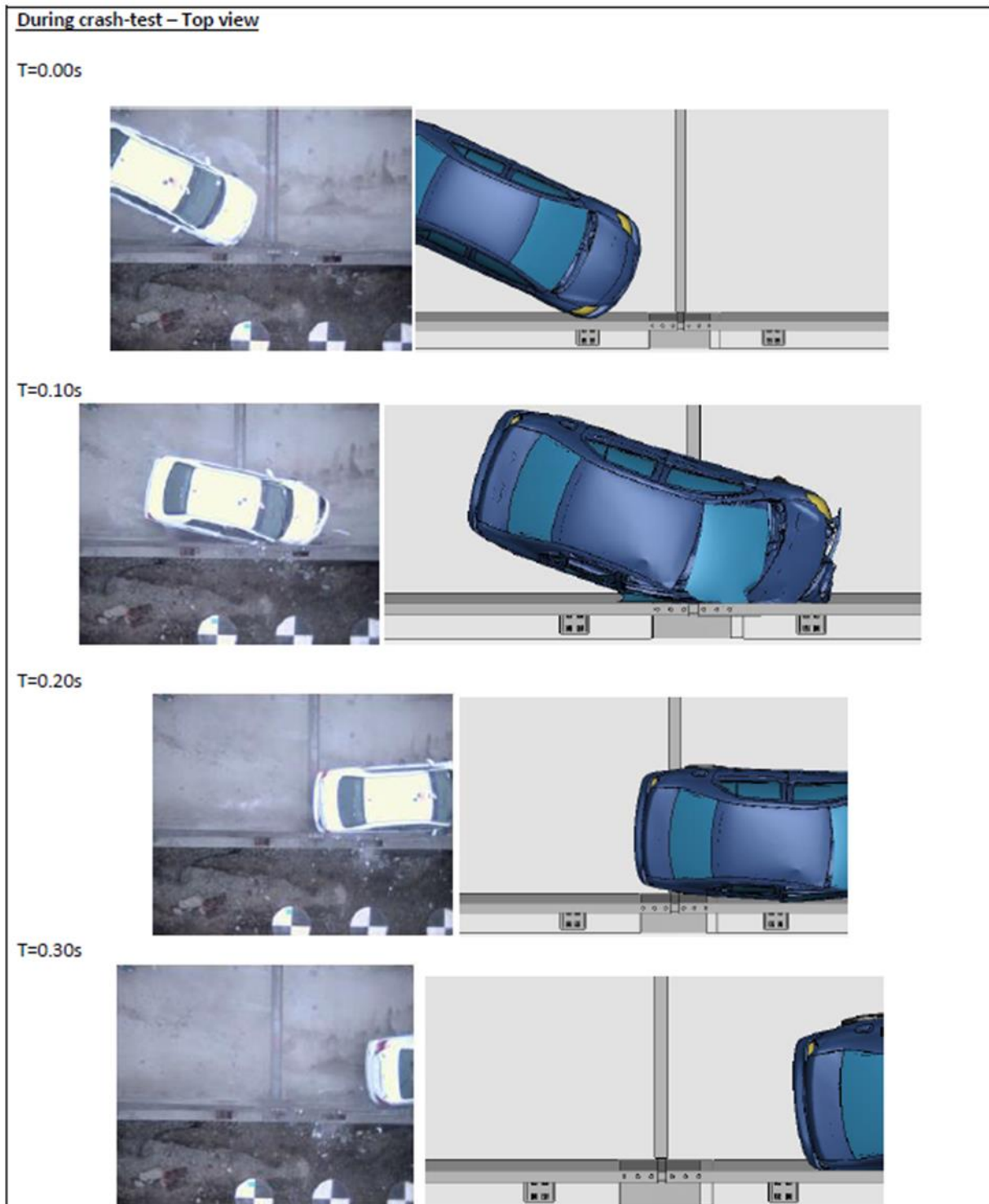
Conclusion

The report concludes that the numerical simulation of the TL5-10, TL5-11 and TL5-12 tests on the TfNSW Medium Performance Type MAO traffic barrier fits the real crash test both in general behaviour of the impact and in performance assessment. GDTech considers the numerical model and simulation presented as validated for the purposes of the study with respect to the TL5-10, TL5-11 and TL5-12 crash test report, according to the MASH and NCHRP179.

The modelling and simulation of the crash test is highly time-consuming, with each simulation potentially taking days to complete. The validation process alone required numerous simulations to adjust the model, resulting in several months of work. Despite this, computational modelling is an invaluable tool for assessing bridge barrier designs, conducting pre-analysis before full crash tests, and analysing modifications to tested bridge barriers once the model is verified and validated.

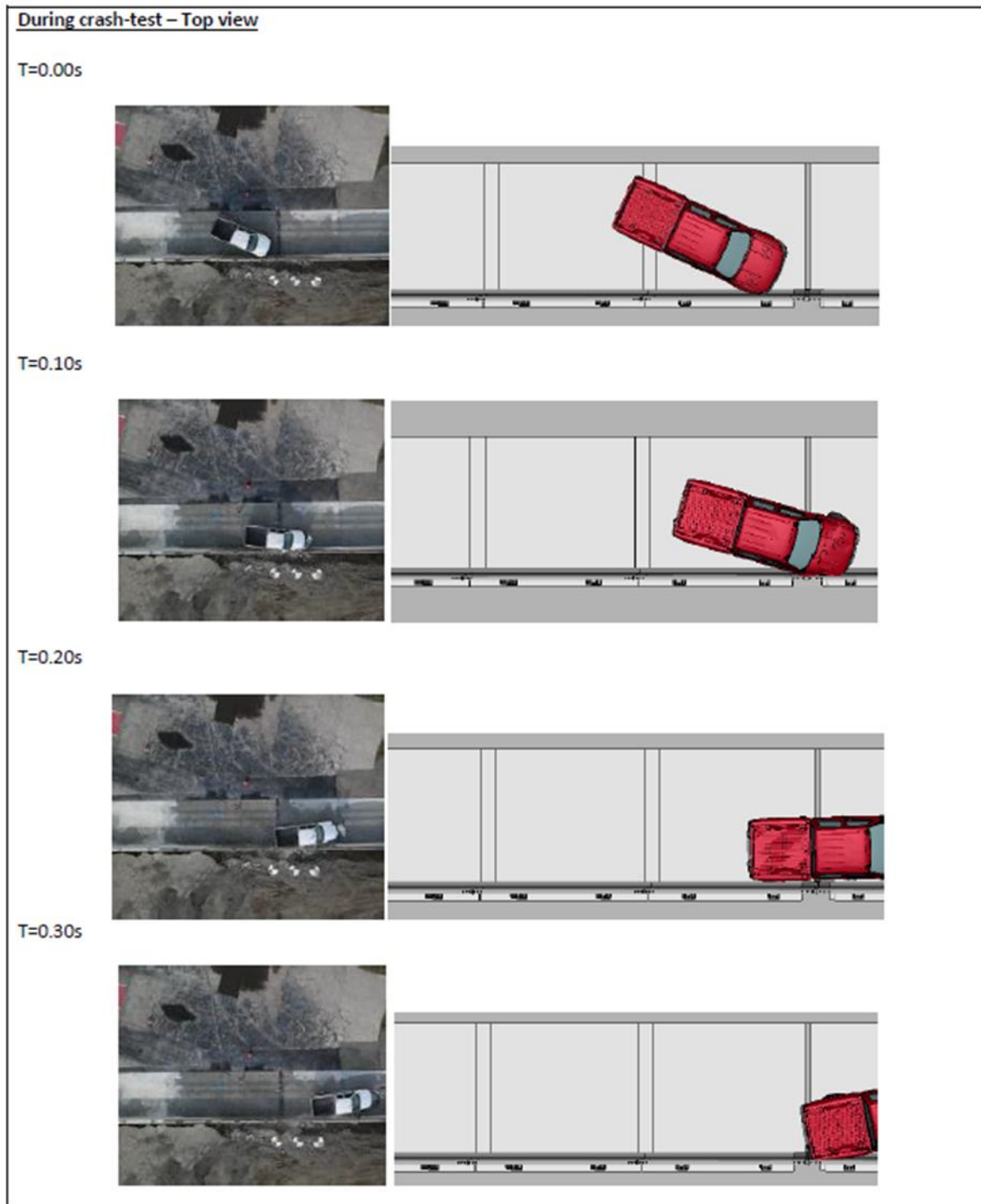
The modelling and simulation require highly specialised software and experienced software users. Experienced engineers are also required who have in-depth technical understanding of the inputs, assumptions, modelling, limitations and outputs to ensure the outcomes are accurate and reflect reality.

Figure 11.5: TL5-10 Side-by-side photographs – top view



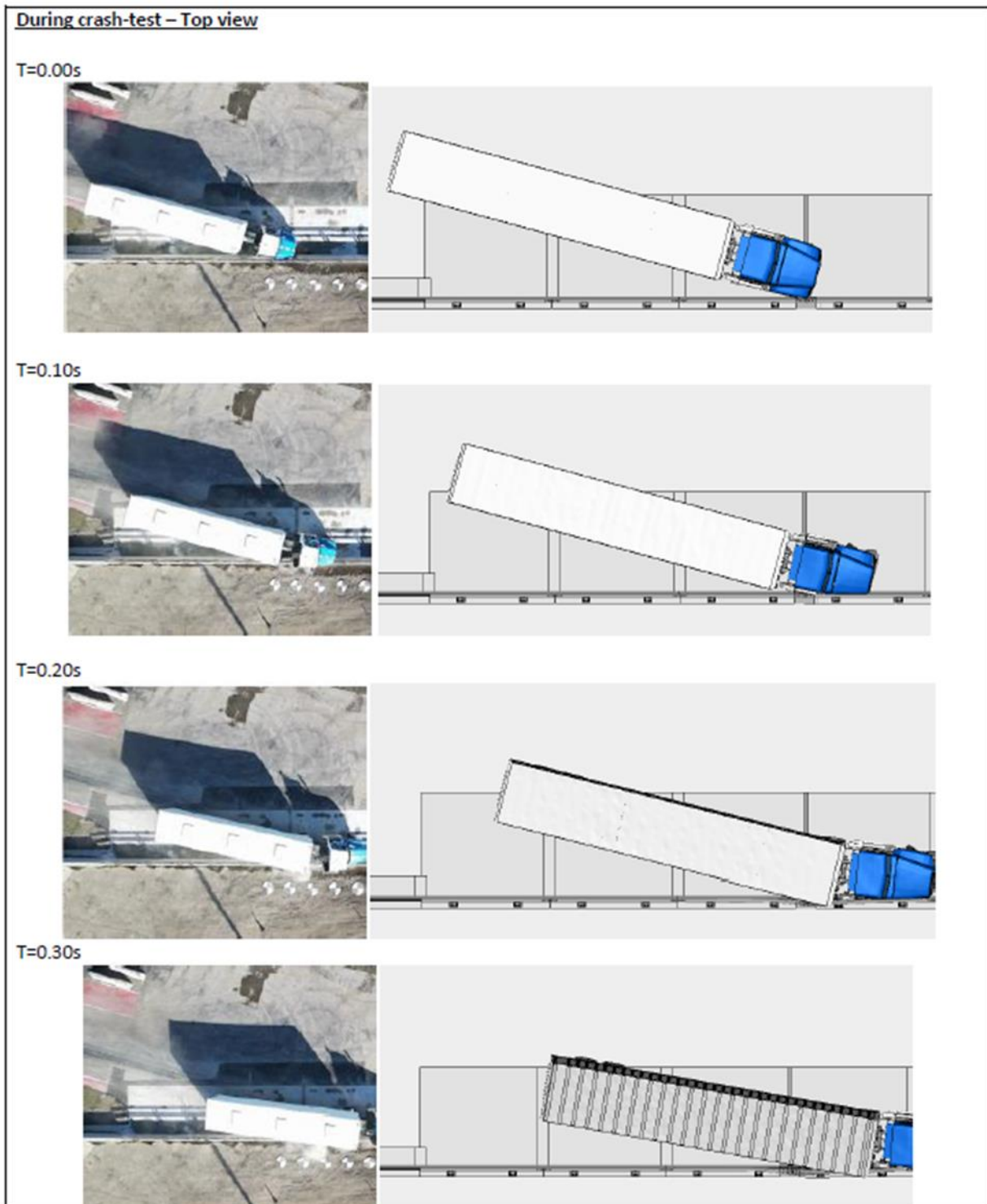
Source: Lucchese and Himpe (2024a).

Figure 11.6: TL5-11 Side-by-side photographs – top view



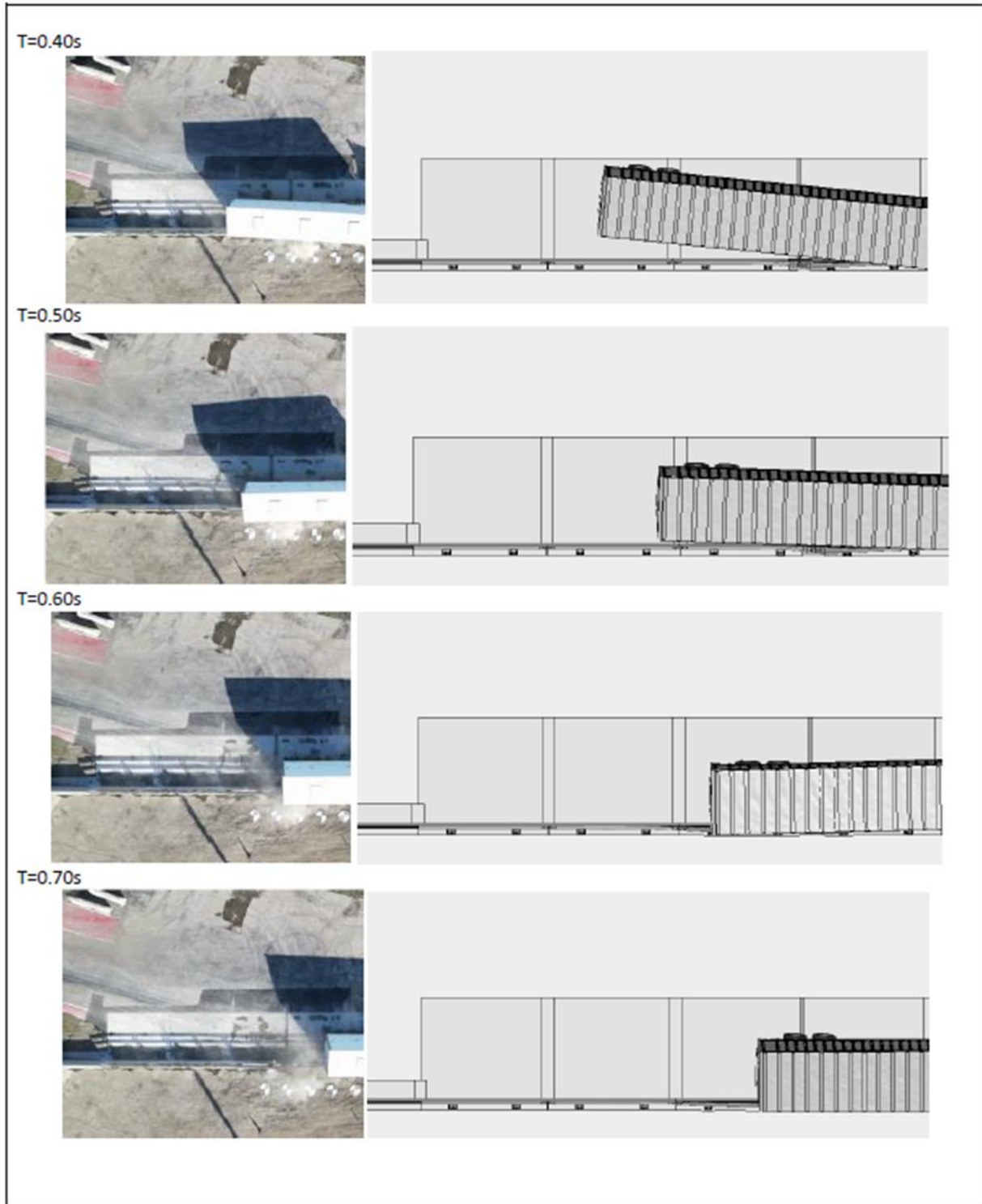
Source: Lucchese and Himpe (2024b).

Figure 11.7: TL5-12 Side-by-side photographs – top view 0 s to 0.3 s



Source: Lucchese and Himpe (2024c).

Figure 11.8: TL5-12 Side-by-side photographs – top view 0.4 s to 0.7 s



Source: Lucchese and Himpe (2024c).

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Appendix A Barrier Design

A.1 Design example 1: Concrete parapet – medium performance level

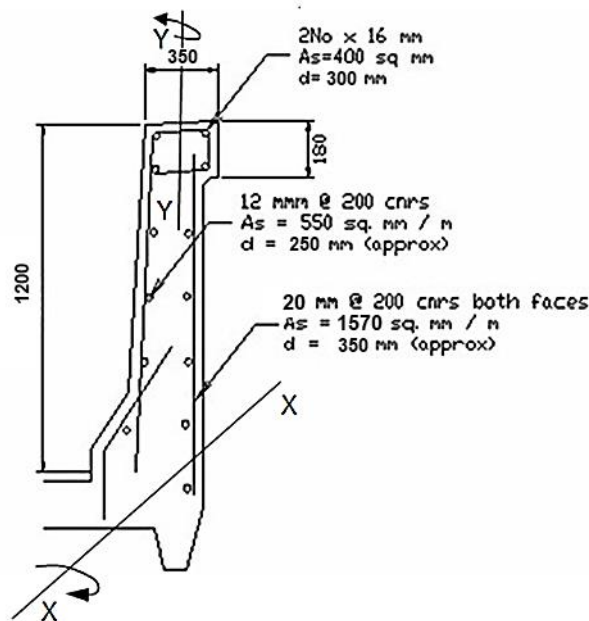
Referring to Table 5.1 for the design forces associated with a ‘medium’ performance level barrier:

- ultimate lateral containment force $F_t = 600 \text{ kN}$
- vehicle contact length $L_t = 2,400 \text{ mm}$
- minimum effective height $H_e = 1,200 \text{ mm}$.

For a concrete parapet to satisfy the minimum effective height requirement, the parapet must have a minimum height of 1,200 mm.

Assume that a parapet-type barrier design shown in Figure A.1 is considered. For this example, the yield strength of steel and characteristic compressive strength of concrete materials are $f_y = 400 \text{ MPa}$ and $f'_c = 32 \text{ MPa}$, respectively.

Figure A.1: Barrier design example 1



Two impact cases need to be considered:

- impact within wall segment
- impact near end wall segment.

A.1.1 Step 1: Calculate the concrete capacities of the various components on the parapet

- Beam at top of wall:

In this example, the beam at the top of the parapet contributes to the capacity of the parapet. The moment capacity of the beam M_b is calculated about the vertical beam axis Y-Y using Equation 7 as follows:

$$M_b = 0.6 \times 400 \times 400 \times 300 \times \left(1 - 0.5 \times \frac{400 \times 400}{0.904 \times 180 \times 300 \times 32}\right) = 27.3 \times 10^6 \text{ N.mm}$$

- Flexural capacity of wall:

The wall flexural capacity M_w is calculated about the vertical axis of the wall Y-Y. The portion of the top beam is not included in the calculation of M_w . For a slope parapet with variable thicknesses along the height, M_w is approximately calculated by dividing the parapet wall into several segments with constant thickness. In this example, it was assumed that the flexural capacity of the wall was constant along its height.

For a unit height of the wall section (Equation 7):

$$M_w = 0.6 \times 550 \times 400 \times 250 \times \left(1 - 0.5 \times \frac{550 \times 400}{0.904 \times 1020 \times 250 \times 32}\right) = 32.5 \times 10^6 \text{ N.mm}$$

- Flexural resistance of cantilevered wall:

The flexural resistance of the cantilevered wall is the wall flexural capacity about the longitudinal axis X-X per metre strip (**Error! Reference source not found.**), using Equation 7:

$$M_c = 0.6 \times 1570 \times 400 \times 350 \times \left(1 - 0.5 \times \frac{1570 \times 400}{0.904 \times 1000 \times 350 \times 32}\right) = 128 \times 10^3 \text{ N.mm/mm}$$

A_s is taken on a strip of one metre.

A.1.2 Step 2: Calculate the critical length of yield line failure pattern, L_c :

- For impact within a wall segment (Equation 4):

$$L_c = \frac{2400}{2} + \sqrt{\left(\frac{2400}{2}\right)^2 + \frac{8 \times 1200(27.3 \times 10^6 + 32.5 \times 10^6)}{128 \times 10^3}} = 3635 \text{ mm}$$

- For impact at the end of wall segment (Equation 6):

$$L_c = \frac{2400}{2} + \sqrt{\left(\frac{2400}{2}\right)^2 + \frac{1200(27.3 \times 10^6 + 32.5 \times 10^6)}{128 \times 10^3}} = 2615 \text{ mm}$$

A.1.3 Step 3: Calculate the wall resistance, R_w :

- For impacts within a wall segment (Equation 3):

$$R_w = \left(\frac{2}{2 \times 3635 - 2400}\right) \left(8 \times 27.3 \times 10^6 + 8 \times 32.5 \times 10^6 + \frac{128 \times 10^3 \times 3635^2}{1200}\right) = 775 \text{ kN}$$

- For impacts at the end of wall segment (Equation 5):

$$R_w = \left(\frac{2}{2 \times 2615 - 2400} \right) \left(27.6 \times 10^6 + 32.6 \times 10^6 + \frac{129 \times 10^3 \times 2615^2}{1200} \right) = 557 \text{ kN}$$

A.1.4 Step 4: Check the structural adequacy of the barrier

From the above calculations, the critical capacity of the barrier is 557 kN for impacts at the end of the wall segment. This value is less than the applied load of $F_t = 600 \text{ kN}$, thus the barrier fails.

Results from a numerical analysis show that the requirement of $R_w \geq F_t$ was satisfied when the vertical reinforcements are $\phi 20 \text{ mm}$ at 150 mm spacings instead of $\phi 20 \text{ mm}$ at 200 mm spacings. Thus $A_s = 2,200 \text{ mm}^2/\text{m}$. The calculations are presented below.

$$M_c = 0.6 \times 2200 \times 400 \times 350 \times \left(1 - 0.5 \times \frac{2200 \times 400}{0.904 \times 1000 \times 350 \times 32} \right) = 177 \times 10^3 \text{ N.mm/mm}$$

- For impacts at the end of wall segment:

$$L_c = \frac{2400}{2} + \sqrt{\left(\frac{2400}{2} \right)^2 + \frac{1200(27.6 \times 10^6 + 32.6 \times 10^6)}{177 \times 10^3}} = 2558 \text{ mm}$$

$$R_w = \left(\frac{2}{2 \times 2558 - 2400} \right) \left(27.3 \times 10^6 + 32.5 \times 10^6 + \frac{177 \times 10^3 \times 2558^2}{1200} \right) = 754 \text{ kN} > F_t \text{ OK}$$

The effective height, Y^* , in the case of a concrete parapet, is equal to the height of the wall:

$$Y^* = 1200 \text{ mm} \geq H_c \text{ OK}$$

The barrier is now adequately designed.

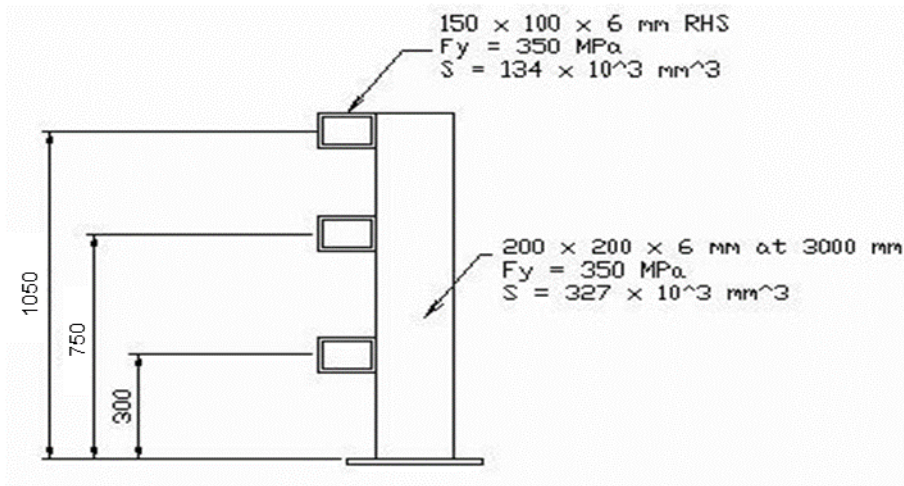
A.2 Design example 2: Post-and-rail barrier – regular performance level

Referring to Table 5.1, for the design forces associated with a ‘regular’ performance level barrier:

- ultimate lateral containment force $F_t = 300 \text{ kN}$
- vehicle contact length $L_t = 1,200 \text{ mm}$
- minimum effective height $H_e = 900 \text{ mm}$.

Assume that a post-and-rail barrier design shown in Figure A.2 is considered with no concrete kerb.

Figure A.2: Barrier design example 2



A.2.1 Step 1: Calculate the ultimate capacities of the post and the rail

Referring to standard section property tables:

150 x 100 x 6 mm RHS has a plastic modulus $S = 134 \times 10^3 \text{ mm}^3$

200 x 200 x 6 mm SHS has a plastic modulus $S = 327 \times 10^3 \text{ mm}^3$

For the top rail (Equation 11),

$$M_p = 0.9 \times 350 \times 134 \times 10^3 = 42.2 \times 10^6 \text{ N.mm}$$

For the middle rail (Equation 11)

$$M_p(\text{middle rail}) = \frac{h_1}{h_2} \times M_p = 30.2 \times 10^6 \text{ N.mm}$$

For the post (Equation 11),

$$M_{p\text{-post}} = \phi f_y S$$

$$M_{p\text{-post}} = 0.9 \times 350 \times 327 \times 10^3 = 103 \times 10^6 \text{ N.mm}$$

The effective height of the barrier, Y^* is calculated using Equation 14, with the assumption that only the 2 top rails contribute to resisting the design load, in order to maximise the effective height of the barrier (**Error! Reference source not found.**):

$$Y^* = \frac{30.2 \times 10^6 \times 750 + 42.2 \times 10^6 \times 1050}{(42.2 \times 10^6 + 30.2 \times 10^6)} = 925 \text{ mm}$$

The shear resistance of the post, P_p , is the shear force on a single post which corresponds to M_p of the post and is located Y^* above the deck and is calculated using Equation 12:

$$P_p = \frac{103.0 \times 10^6}{925} = 111 \times 10^3 \text{ N}$$

A.2.2 Step 2: Calculate the barrier resistance

The critical barrier nominal resistance shall be taken as the least value of the following, with an assumption that the bottom rail does not contribute to resisting the design load:

- For failure modes involving an odd number of railing spans, N (Equation 9):

For $N = 1$:

$$R^* = \frac{16 \times (42.2 \times 10^6 + 30.2 \times 10^6) + 0}{2 \times 1 \times 3000 - 1200} = 241 \text{ kN}$$

For $N = 3$:

$$R^* = \frac{16 \times (42.2 \times 10^6 + 30.2 \times 10^6) + (3 - 1)(3 + 1) \times 114 \times 10^3 \times 3000}{2 \times 3 \times 3000 - 1200} = 228 \text{ kN}$$

For $N = 5$:

$$R^* = \frac{16 \times (42.2 \times 10^6 + 30.2 \times 10^6) + (5 - 1)(5 + 1) \times 114 \times 10^3 \times 3000}{2 \times 5 \times 3000 - 1200} = 319 \text{ kN}$$

- For failure modes involving an even number of railing spans, N (Equation 10):

For $N = 2$:

$$R^* = \frac{16 \times (42.2 \times 10^6 + 30.2 \times 10^6) + 2^2 \times 114 \times 10^3 \times 3000}{2 \times 2 \times 3000 - 1200} = 231 \text{ kN}$$

For $N = 4$:

$$R^* = \frac{16 \times (42.2 \times 10^6 + 30.2 \times 10^6) + 4^2 \times 114 \times 10^3 \times 3000}{2 \times 4 \times 3000 - 1200} = 285 \text{ kN}$$

For $N = 6$:

$$R^* = \frac{16 \times (42.2 \times 10^6 + 30.2 \times 10^6) + 6^2 \times 114 \times 10^3 \times 3000}{2 \times 6 \times 3000 - 1200} = 378 \text{ kN}$$

A.2.3 Step 3: Check the structural adequacy of the barrier

From the above cases, it is clear that the capacity of the barrier is controlled by the failure mode involving 3 railing spans ($N = 3$) with a capacity of 228 kN. This does not meet the capacity requirement ($R^* \geq F_t = 300 \text{ kN}$).

Results from a numerical analysis show that the requirement of $R^* \geq F_t$ was satisfied when the horizontal rails were increased to 150 x 100 x 8 mm sections ($S = 169,000 \text{ mm}^3$) and the posts were increased to 200 x 200 x 8 mm sections ($S = 421,000 \text{ mm}^3$). The calculations are presented below.

Referring to standard section property tables:

150 x 100 x 6 mm RHS has a plastic modulus $S = 169 \times 10^3 \text{ mm}^3$

200 x 200 x 6 mm SHS has a plastic modulus $S = 421 \times 10^3 \text{ mm}^3$

For the top rail (Equation 11)

$$M_p = 0.9 \times 350 \times 169 \times 10^3 = 53.2 \times 10^6 \text{ N.mm}$$

For the middle rail (Equation 11)

$$M_p(\text{middle rail}) = \frac{h_1}{h_2} \times M_p = 38.0 \times 10^6 \text{ N.mm}$$

For the post (Equation 11)

$$M_{p\text{-post}} = 0.9 \times 350 \times 421 \times 10^3 = 133 \times 10^6 \text{ N.mm}$$

The effective height of the barrier, Y^* will remain 925 mm as both top rails have the same plastic modulus ($S = 169,000 \text{ mm}^3$) and heights.

The shear resistance of the post, P_p , is the shear force on a single post which corresponds to M_p of the post and is located Y^* above the deck. It is calculated using Equation 12:

$$P_p = \frac{133 \times 10^6}{925} = 143 \times 10^3 \text{ N}$$

The critical barrier nominal resistance for 3 spans ($N = 3$):

For $N = 3$:

$$\begin{aligned} R^* &= \frac{16 \times (53.2 \times 10^6 + 38.0 \times 10^6) + (3 - 1)(3 + 1) \times 147 \times 10^3 \times 3000}{2 \times 3 \times 3000 - 1200} \\ &= 306 \text{ kN} \geq F_t = 300 \text{ kN OK} \end{aligned}$$

The computed effective height can now be found using Equation 14 as:

$$\begin{aligned} R_i(\text{Top rail}) &= \frac{306}{1 + \frac{750}{1050}} = 179 \text{ kN} \\ R_i(\text{Middle rail}) &= \frac{306 * \frac{750}{1050}}{1 + \frac{750}{1050}} = 127 \text{ kN} \\ Y^* &= \frac{127 * 750 + 179 * 1050}{(127 + 179)} = 925 \text{ mm} \geq H_e \quad \text{OK} \end{aligned}$$

The barrier is now adequately designed.

A.3 Design Example 3: Combined concrete parapet and metal rail – medium performance level

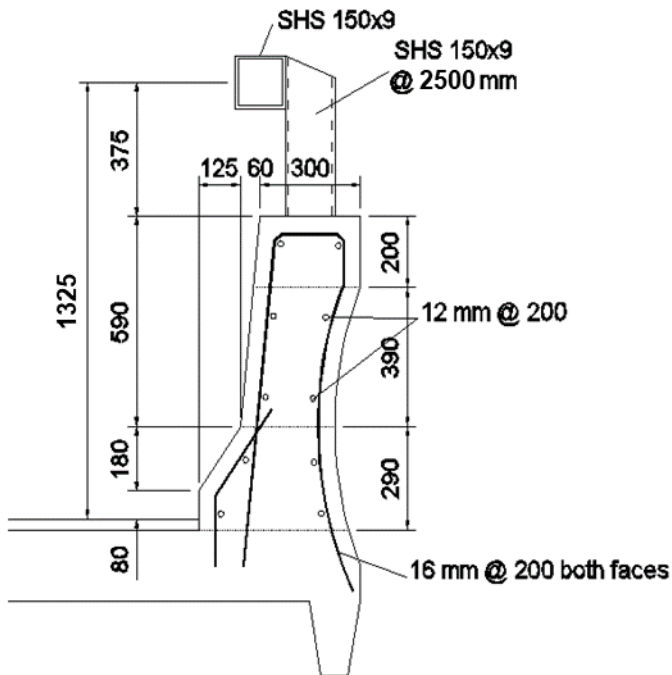
Referring to Table 5.1, for the design forces associated with a 'medium' level barrier:

- ultimate lateral containment force $F_t = 600 \text{ kN}$
- vehicle contact length $L_t = 2,400 \text{ mm}$
- minimum effective height $H_e = 1,200 \text{ mm}$.

The barrier must have sufficient strength to resist the ultimate lateral containment force, 600 kN at the required minimum effective height of 1,200 mm.

Assume that a combined concrete parapet and metal rail barrier design shown in Figure A.3 is considered.

Figure A.3: Barrier design example 3 – barrier cross-section



A.3.1 Step 1: Calculate the ultimate capacities of the various components

- Concrete parapet

For this example, the following material properties are used:

Yield strength of reinforcement $f_y = 400 \text{ MPa}$

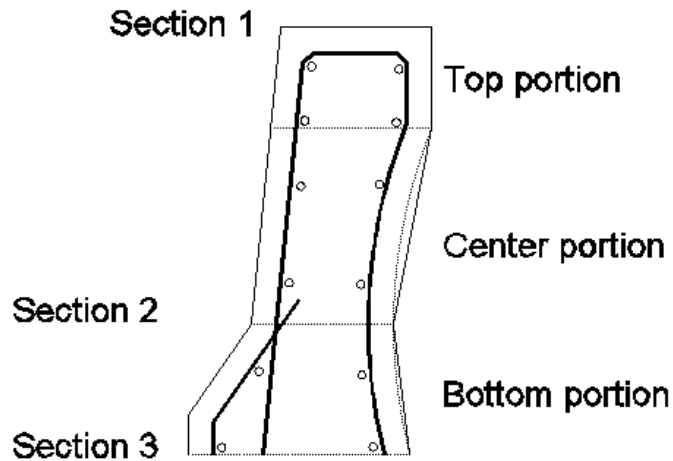
Characteristic compressive strength of concrete at 28 days $f'_c = 25 \text{ MPa}$

Strength reduction factor for concrete $\phi_c = 0.8$

There is no concrete beam at the top of the wall, thus $M_b = 0$.

As the thickness of the concrete parapet varies along its height, it is assumed that the parapet is divided into 3 portions as shown in Figure A.4. Two possible failure mechanisms shown in Figure 5.6 are considered.

Figure A.4: Barrier design example 3 – parapet



The flexural resistance of a cantilevered wall, M_c is calculated as the average of the values of M_c at sections 1, 2 and 3 as follows:

Section 1: At top of wall	Symbol	Value	Unit
Height of cross-section		300	mm
Cover thickness of vertical rebars		50	mm
Diameter of vertical rebars		16	mm
Spacing of vertical rebars	b	200	mm
Cross-section area of one vertical rebar	A_s	201	mm ²
Height of top segment	H_1	590	mm
Effective height of cross-section	d	242	mm

Moment capacity of cantilevered wall at section 1:

$$M_c = \phi A_s f_y d \left(1 - 0.5 \frac{A_s f_y}{\alpha_2 b d f'_c} \right) / b$$

M_{cs1}	75.1×10^3	Nmm/mm
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Section 2: At bottom of top slope	Symbol	Value	Unit
Height of cross-section		300	mm
Cover thickness of vertical rebars		50	mm
Diameter of vertical rebars		16	mm

Spacing of vertical rebars	b	200	mm
Cross-section area of one vertical rebar	A _s	201	mm ²
Effective height of cross-section	d	242	mm

Moment capacity of cantilevered wall at section 2:

$$M_c = \phi A_s f_y d \left(1 - 0.5 \frac{A_s f_y}{\alpha_2 b d f'_c}\right) / b$$

M _{cs2}	75.1 x 10 ³	Nmm/mm
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Section 3: At bottom of wall

Symbol	Value	Unit
Height of cross-section	485	mm
Cover thickness of vertical rebars	50	mm
Diameter of vertical rebars	16	mm
Spacing of vertical rebars	b	200
Cross-section area of one vertical rebar	A _s	200
Height of bottom segment	H ₂	260
Effective height of cross-section	d	425

Moment capacity of cantilevered wall at section 3:

$$M_c = \phi A_s f_y d \left(1 - 0.5 \frac{A_s f_y}{\alpha_2 b d f'_c}\right) / b$$

M _{cs3}	135 x 10 ³	Nmm/mm
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Assuming failure mechanism 1, M_c is the average of the 3 sections:

$$M_c = \left(\frac{M_{cs1} + M_{cs2}}{2} H_1 + \frac{M_{cs2} + M_{cs3}}{2} H_2 \right) \left(\frac{1}{H_1 + H_2} \right)$$

M _{c1}	84.2 x 10 ³	Nmm/mm
-----------------	------------------------	--------

Assuming failure mechanism 2, M_c is the average of the top 2 sections:

$$M_c = \left(\frac{M_{cs1} + M_{cs2}}{2} \right)$$

M _{c2}	75.1 x 10 ³	Nmm/mm
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The flexural capacity of the wall, M_w is calculated separately for positive bending and negative bending for each portion. The total flexural capacity of the wall is the sum of the average capacities of the number of wall portions in association with each failure mechanism.

Top portion	Symbol	Value	Unit
Width at top section		300	mm
Width at bottom section		320	mm
Height of portion	H _p	200	mm
Distance from outer fibre to centre of tension rebar		76	mm
Diameter of longitudinal rebar		12	mm
Cross-section area of one rebar		113	mm ²
Number of longitudinal rebars in positive moment		2	
Number of longitudinal rebars in negative moment		2	
Cross-section area of rebar (same for both positive and negative)	A _s	226	mm ²
Average effective height of wall section	d	234.2	mm

Moment capacity of wall M_w of top portion

$$M_{\text{positive}} = M_{\text{negative}}$$

$$M_{\text{positive}} = M_{\text{negative}} = \phi A_s f_y d \left(1 - 0.5 \frac{A_s f_y}{\alpha_2 b d f'_c}\right)$$

$$M_w = 16.2 \times 10^6 \text{ Nmm}$$

Centre portion

	Name	Value	Unit
Width at top section		320	mm
Width at bottom section		320	mm
Height of portion	H _p	390	mm
Diameter of longitudinal rebar		12	mm
Cross-section area of 1 rebar		113	mm ²
Distance from outer fibre to centre of rebar for positive moment		76	mm
Number of longitudinal rebars in positive moment		2	

Number of longitudinal rebars in negative moment		2	
Cross-section area of rebars (same for both positive and negative)	A_s	226	mm ²
Average effective height of wall section	d	244.2	mm

Moment capacity of wall M_w

$$M_{positive} = M_{negative} = \phi A_s f_y d \left(1 - 0.5 \frac{A_s f_y}{\alpha_2 b d f'_c}\right)$$

M_w	17.3×10^6	Nmm
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Bottom portion

	Name	Value	Unit
Width at top section		320	mm
Width at bottom section		485	mm
Height of portion	H_p	260	mm
Diameter of longitudinal rebar		12	mm
Cross-section area of one rebar		113	mm ²
Distance from outer fibre to centre of rebar for positive moment		76	mm
Number of longitudinal rebars in positive moment		2	
Number of longitudinal rebars in negative moment		2	
Cross-section area of rebars (same for positive and negative moment)	A_s	226	mm ²
Average effective height of wall section	d	326.5	mm

Moment capacity of wall M_w

$$M_{positive} = M_{negative} = \phi A_s f_y d \left(1 - 0.5 \frac{A_s f_y}{\alpha_2 b d f'_c}\right)$$

M_w	23.1×10^6	Nmm
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For collision within a segment, the average of the positive and negative values of the wall's flexible capacity is used. This is acceptable because the yield line mechanism for this case will have some positive moment hinges and some negative moment hinges.

For collision near expansion joints, the moment resistance that corresponds to a moment causing tension along the inside face of the parapet, i.e. the positive moment, is used. This is required because the only yield line to form is caused by a moment causing tension along the inside face.

The parapet combined capacity, R_w , is the smaller value of 2 mechanisms, calculated as follows.

Collision within a segment:

Failure mechanism 1 (all 3 portions)	M_w	56.6×10^6	Nmm
Length of critical yield line (Equation 4)	L_c	3652	mm
Total transverse resistance of barrier (Equation 3)	R_w	723	kN
Failure mechanism 2 (2 top portions)	M_w	33.5×10^6	Nmm
Height of failure mechanism 2	H_1	590	mm
Length of critical yield line (Equation 4)	L_c	3,083	mm
Total transverse resistance of barrier (Equation 3)	R_w	785	kN

Therefore, $R_w = 723$ kN for collision within a segment.

Collision near an end segment:

Failure mechanism 1			
M_w is the sum of $M_{positive}$ in three portions	M_w	56.6×10^6	Nmm
Length of critical yield line (Equation 6)	L_c	2,618	mm
Total transverse resistance of barrier (Equation 5)	R_w	518	kN
Failure mechanism 2			
Height of failure mechanism 2	H_1	590	mm
M_w is the sum of $M_{positive}$ in two top portions	M_w	33.5×10^6	Nmm
Length of critical yield line (Equation 6)	L_c	2,505	mm
Total transverse resistance of barrier (Equation 5)	R_w	637	kN

Therefore, $R_w = 518$ kN for collision near an end segment.

- Metal rail-and-post

Referring to standard section property tables:

150 x 150 x 9 mm SHS has a plastic modulus, $S = 248 \times 10^3 \text{ mm}^3$

The rail and post have the same section, and the ultimate capacity is (Equation 11):

$$M_p = 0.9 \times 450 \times 248 \times 10^3 = 100 \times 10^6 \text{ N.mm}$$

The resistance of the post, P_p is calculated using Equation 12 as:

$$P_p = \frac{100 \times 10^6}{1325 - 850} = 211 \times 10^3 \text{ N}$$

For impact involving one railing span, $N = 1$ (Equation 9):

$$R_r = \frac{16 \times 100 \times 10^6 + 0}{2 \times 1 \times 2500 - 2400} = 618 \text{ kN}$$

For impact involving 2 railing spans, $N = 2$ (Equation 10):

$$R'_r = \frac{16 \times 100 \times 10^6 + 2^2 \times 211 \times 10^3 \times 2500}{2 \times 2 \times 2500 - 2400} = 490 \text{ kN}$$

A.3.2 Step 2: Calculate the resistance of the combination parapet and rail

- For impact at mid-span of the metal rail (one span failure):

Resistance of the parapet	R_w	723	kN
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Resistance of the metal rail over one span	R_r	618	kN
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Resistance of combined barrier (Equation 23)	R^*	1,341	kN
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Check the resistance of barrier to transverse design load:

$R^* \geq F_t?$	YES	600	kN
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Effective height of barrier (Equation 22)	Y^*	1,068	mm
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Check the effective height of barrier:

$Y^* \geq H_e?$	NO	1,200	mm
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In this case the effective height of the barrier, Y^* , does not meet the requirement of $Y^* \geq H_e$. However, because the total barrier resistance is 1,341 kN which is significantly greater than the applied load of $F_t = 600 \text{ kN}$, Y^* can be increased by reducing the contribution of the concrete parapet R_w with a corresponding reduction in the total lateral resistance, R^* .

Using 20% of R_w ; the barrier's total resistance and effective height are re-calculated as follows:

Revised resistance of the metal rail over one span	R_w	144	kN
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Revised resistance of combined barrier (Equation 19)	R^*	763	kN
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Check the resistance of barrier to transverse design load:

$R^* \geq F_t?$	YES	600	kN
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Effective height of barrier (Equation 20)	Y^*	1,235	mm
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Check the effective height of barrier:

$Y^* \geq H_e?$	YES	1,200	mm
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- For impact at a post (2 span failure):

Resistance of the post	P_p	211	kN
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Resistance of the metal rail over 2 spans	R'_r	490	kN
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Resistance of the parapet, reduced to resist the post load (Equation 23)	R'_w	404	kN
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Resistance of combined barrier (Equation 21)	R^*	1,105	kN
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Check the resistance of barrier to transverse design load:

$R^* \geq F_t?$	YES	600	kN
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Effective height of barrier (Equation 22)	Y^*	1,151	mm
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Check the effective height of barrier:

$Y^* \geq H_e?$	NO	1,200	mm
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Again, the effective height of the barrier, Y^* , does not meet the requirement of $Y^* \geq H_e$. However, because the barrier's total resistance is significantly greater than the applied load of $F_t = 600$ kN, Y^* can be increased by reducing the contribution of the concrete parapet R_w with a corresponding reduction in the total lateral resistance, R^* .

Using 75% of R_w , the barrier's total resistance and effective height are re-calculated as follows:

Revised resistance of the metal rail over 2 spans	R_w	542	kN
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Resistance of the parapet, reduced to resist the post load (Equation 23)	R'_w	213	kN
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Resistance of combined barrier (Equation 21)	R^*	914	kN
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Check the resistance of barrier to transverse design load:

$R^* \geq F_t?$	YES	600	kN
Effective height of barrier (Equation 22)	Y^*	1,214	mm
Check the effective height of barrier:			
$Y^* \geq H_e?$	YES	1,200	mm

From the above calculations, the barrier is adequately designed.

Appendix B Retrofit of Barrier

This section provides suggestions and guidance on the retrofitting of traffic barriers on existing bridges, taking into account the design loads and barrier performance levels specified in AS 5100.1:2017, the risk assessment and barrier selection criteria.

This section covers the following:

- evaluation of an existing bridge structure
- barrier retrofit design.

A standard set of designs for retrofitting existing bridges barriers is not possible due to the designs being site-specific and highly variable.

B.1 Evaluation requirements

Once the appropriate barrier performance level has been selected the bridge needs to be evaluated to determine if this barrier performance level is achievable. This consists of a review and calculations to determine:

- the logistics of demolition of the old barrier and constructing an appropriate barrier on the existing structure
- if the existing structure is adequate to carry the higher loads associated with the appropriate barrier; this needs to include an assessment of the
 - transverse deck capacity
 - girder capacity
 - clearance for placement of the new barrier
 - other relevant requirements.

Details of the slab and kerb reinforcement of the existing bridge need to be reviewed to determine if the slab edge is capable of being retrofitted with an adequate new barrier. The following information should be acquired to perform the deck evaluation:

- as-built plan of the whole structure including detailed design drawings of the bridge's superstructure components
- a plot locating existing conditions on the slab such as delamination, spalls and cracks
- measurement of the depth of cover on the top mat of reinforcing steel on a grid pattern
- deck concrete compressive strength assessed through destructive testing of deck core samples.

B.2 Evaluation methodology

It is assumed that there is a section of deck slab cantilever that needs to transfer the barrier collision loads to the exterior beam. Due to the extreme nature of barrier collisions and the low probability of occurrence, a simplified yield line method based on AASHTO (2020) is adopted (Section 5). Inherent in the use of yield line theory is the assumption that the historic mild steel reinforcement used for bridge construction has sufficient ductility to allow the yield line structural response to develop. Refer to Section 5.4 for post-and-railing systems and Section 5.3 for barriers with a concrete parapet.

In addition, AS 5100.2:2017 requires that the bridge barriers and supporting elements be designed as a progressive strength system in which barriers and then their connections fail prior to the failure of the bridge deck slab. Therefore, a load factor of 1.1 should be used when evaluating the deck slab cantilever section under the above impact load combinations.

AS 5100.8, which was added to the AS 5100 series in 2017, identifies the process for designing a replacement traffic barrier for bridges. A risk assessment should be undertaken in accordance with AS 5100.1:2017 to identify the required performance level. If the bridge cannot be economically strengthened based on the performance level identified, the highest barrier performance level within the economic constraints should be adopted. Where the performance level is less than required from the risk assessment, alternative strategies should be considered, including reduced speed limits and advanced warning signs.

B.3 Load combinations

While barrier collision loads are known for a predefined barrier performance level, the specific bridge loading at the time of the collision is less certain. The barrier impact and traffic loads on the deck need not be applied simultaneously when designing the deck. For the purpose of assessing the capacity of the existing deck due to barrier impacts, the following 2 load combinations need to be considered:

- Case 1: Permanent loads + the transverse and longitudinal impact forces
- Case 2: Permanent loads + the vertical impact forces.

The impact forces are taken from the barrier design loads for the performance level under consideration (Section 3.1).

Appendix C Benefit Cost Analysis

C.1 Available quantitative assessment methods

There are 4 methods that may be used to undertake a quantitative assessment of the risk associated with a hazard. Jurisdictions in Australia and New Zealand may use one or several of these methods depending on the complexity of the hazards. The methods available are:

- simple manual method
- detailed quantitative manual method
- Queensland Department of Transport and Main Roads – *TMR Roadside Impact Severity Calculator (RISC)* (Queensland Department of Transport and Main Roads 2021)
- *AASHTO Roadside safety analysis program (RSAP)* (National Academies of Sciences, Engineering, and Medicine 2022).

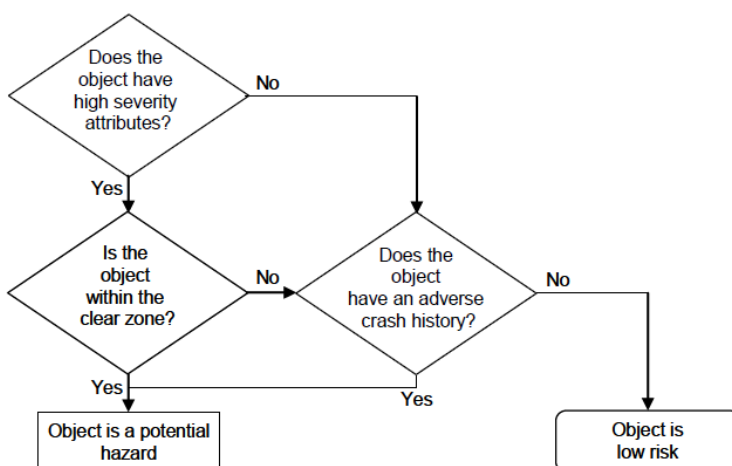
These methods are briefly described as follows.

1. Simple manual method.

This method involves (Figure C.1):

- an assessment of the likelihood that a vehicle will crash into an object or feature and the severity of such crashes
- an assessment of the number of crashes that would occur with a treatment and the severity of those crashes
- a comparison of the number and severity of crashes that would occur at an untreated hazard with the number and severity of crashes if a treatment was provided (e.g. road safety barrier).

Figure C.1: Simple process for risk assessment



Source: Based on Queensland Department of Main Roads (2021).

This method is usually used for hazard identification in a risk assessment process.

2. Detailed quantitative manual method

The detailed manual method is based on a risk assessment and economic analysis. The process involves:

- determining the annual crash cost for the hazard from crash data for an existing situation
- determining options for treatment of the hazard
- estimating costs associated with each treatment option including crash costs, construction costs, annual maintenance costs and operating costs where applicable
- undertaking a B/C analysis of the whole-of-life costs for the existing situation (i.e. the untreated hazard) and of all treatment options identified for evaluation and consideration
- using the results of the B/C analysis in conjunction with any qualitative analysis to establish which treatment was recommended.

For bridge barriers, treatment options may include barriers of different performance levels. The assessment will compare the BCRs when providing different performance level barriers for the bridge site, based on the specific site conditions.

3. TMR RISC

RISC is a program developed by the TMR based on the AASHTO software *ROADSIDE* and is used to perform quantitative evaluation of hazardous roadside objects (Queensland Department of Transport and Main Roads 2021).

RISC requires the user to model roadside objects and potential treatments for these objects using an array of numerical parameters. Once this is done the relative benefits and costs for different treatments are automatically calculated using an algorithm based on the *AASHTO Roadside Design Guide* (AASHTO 2022). The most cost-effective treatment for each hazard can be determined and the decision-making process can then continue to the next step. The program operates through a series of windows and menus.

The modelling can be used to determine the possible BCRs achievable by comparing the treatment options available. For example, a comparison can be made between leaving an end-on culvert as it is, installing bar grates, redesigning the end wall to reduce its severity, and the installation of a road safety barrier.

TMR does not use this method specifically for bridge barriers, however, the principles of the B/C analysis method used in this program may be utilised.

4. AASHTO RSAP

Road Safety Analysis Program (RSAP) software is described in Appendix A of the *Roadside Design Guide* (AASHTO 2006) and represents one approach to using this guide. RSAP provides an improved computer-based cost-effectiveness analysis procedure for use in:

- assessing alternative roadside safety treatments at both point locations and for sections of roadway
- developing warrants and guidelines including those which consider performance levels of safety features.

RSAP provides a simple and structured means for data entry and 4 separate reports summarising the analysis results and input data, namely:

- BCR – presents the incremental B/C ratios associated with the alternatives in a tabular format for all combinations of alternatives.
- Alternative cost – presents the predicted crash frequencies, and the annual installation, maintenance and repair costs associated with each of the alternatives in a tabular format.

- Feature cost – presents the predicted impact frequencies, average severity and crash costs associated with individual features of each alternative in a tabular format.
- Input data – presents the input data for each alternative in a summary form.

C.2 Proposed B/C analysis method

The quantitative evaluation processes used by various road jurisdictions and computer programs are similar but the detail of some parameters may vary to suit the requirements of the jurisdiction. The methods involve a hazard risk assessment and a typical process should include:

- Step 1 – calculation of the frequency of errant vehicle crashes
- Step 2 – determination of the severity of the crashes with the hazard/s
- Step 3 – comparison of the results for a hazard with those for a treatment.

The information presented below makes use of the principles of the evaluation procedure used by the RISC software (Department of Main Roads 2005) to describe the methodology for calculating *BCRs*.

The benefit cost ratio (BCR) is defined as the net present benefit (NPB) divided by the net present cost (NPC) as (Equation A 1):

$$BCR = \frac{NPB}{NPC} \quad A\ 1$$

where

NPB = the total value of benefits due to crash reduction over a defined period based on an economic discount rate, calculated as:

$$NPB = (factor) \times B$$

where (factor) is a discounting factor, for different values of rate and period; B is the value of annual benefits (i.e. annual reduction in road crash cost)

NPC = the cost of implementation (discounted if not undertaken in the first year).

The following factors are considered in determining a *BCR*:

- cost savings in crashes prevented or reduced severity (reduction in road crash cost)
- cost of implementing treatment
- cost of maintaining treatment
- cost of repairing treatment if hit
- length of analysis period
- discount rate.

The *BCR* will be checked to compare the cost of barrier installation for a given performance level against the probable crash cost for the design life. This allows assessment of different barrier performance levels and use of a higher crash cost to reflect the consequences to third parties.

A *BCR* > 1 is considered the minimum criterion for installing roadside barriers with a higher performance level. TMR uses the criterion of a *BCR* greater than 1.5 for rural roads and 2.5 for urban roads. VicRoads uses a *BCR* of 4.0 or higher.

Figure C.2 presents a flowchart for determining the crash cost in a risk assessment process based on the procedure used by the RISC software (Queensland Department of Transport and Main Roads 2024).

The steps in the flowchart are described in detail below:

1. Determine road environment variables

These variables define the roadway characteristics and are used to determine the base encroachment frequency (the number of expected encroachments per km per year). The road environment variables include:

- road type: divided, undivided and one-way
- number of lanes (on each carriageway)
- width of lanes (marked lanes)
- design speed = 85th percentile speed or the posted speed limit +10 km/h if the data is not available.

The following roadside object attributes, in combination with vehicle speed and road curvature, which define the probability of impact:

- horizontal offset of the object from the edge of the travelled way. This should be calculated for each lane as the distance from the edge of each lane to the inner face of the barrier
- object length is the length of the bridge barrier
- object width. For bridge barriers as a roadside object, the width of barrier is not considered as the vehicle impact to the barrier's back and ends is unlikely to occur
- determine traffic data.

The traffic volume is obtained from traffic survey counts and/or available reports. The following information is derived from the traffic data:

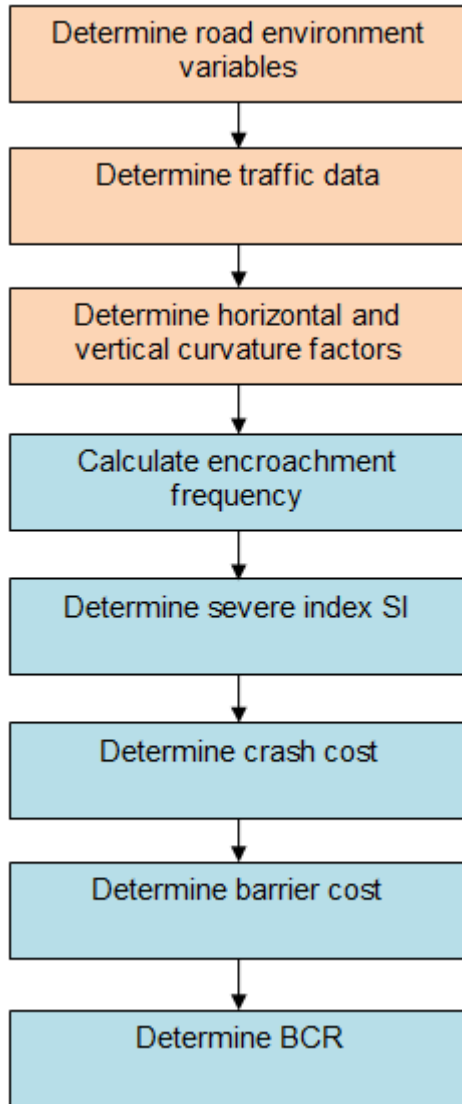
- AADT: total estimated average annual daily traffic. AADT is taken from the project's EIS (environmental impact statement). A traffic growth rate, design life and maximum lane capacity should be provided to calculate the average AADT over the design life
- growth rate p.a. (for example, Queensland Department of Transport and Main Roads (2005) uses 4%, AS 5100.1:2004 uses 2%)
- design life
- maximum lane capacity
- the percentage of commercial vehicles.

The traffic volume is divided into the number of carriageways. For example, on a two-lane two-way road, the traffic volume would remain unchanged (i.e. it is a single carriageway), whereas for a four-lane divided facility, the volume is divided by 2 for a 50%/50% split unless another split of traffic is evident (Queensland Department of Transport and Main Roads 2005). It should be noted that this AADT is different from the AADT used in the chart-based procedure, which is the AADT at the construction year.

2. Determine horizontal and vertical curvature factors

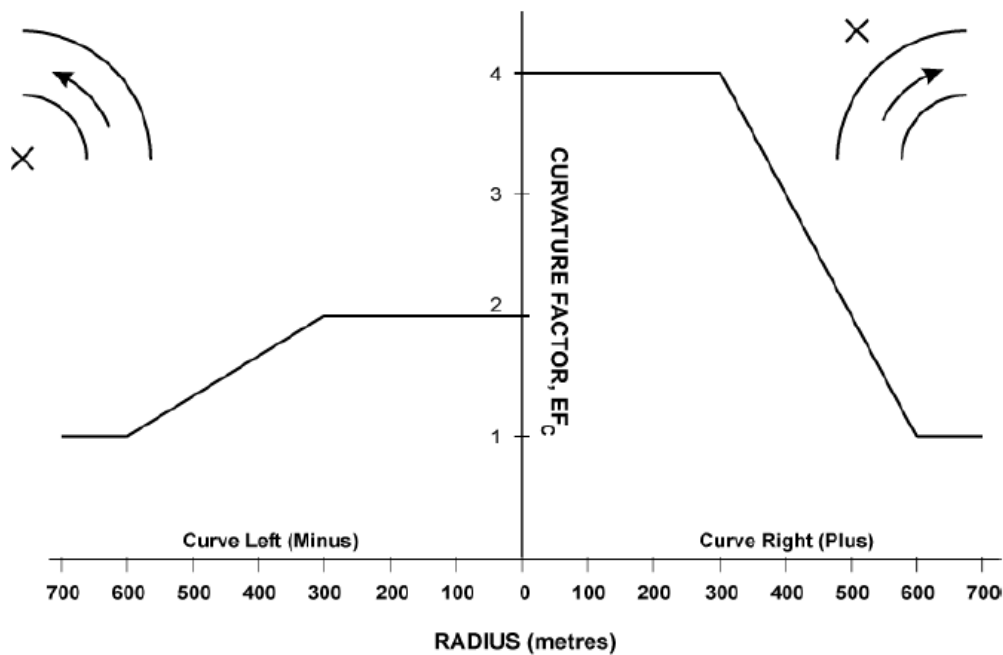
These factors modify the encroachment frequency, due to the increased probability of a vehicle leaving the road on horizontal curves or grades. Note that the grade adjustment factor in Queensland Department of Transport and Main Roads (2005) is the same as that of AS 5100.1:2004 (Figure 3.3); however the curvature factor is different (Figure C.3).

Figure C.2: Flowchart of risk assessment process



Source: Based on Queensland Department of Transport and Main Roads (2005).

Figure C.3: Curvature adjustment factor CU



Source: Queensland Department of Transport and Main Roads (2005).

3. Calculate encroachment frequency

This is an estimation of the number of vehicles that will leave the roadway per km per year. Not all vehicles that leave the roadway will necessarily collide with a roadside object. Variables such as the object's size and offset from the edge of the travelled way and vehicle speed influence the likelihood of impact with the object. The encroachment frequency is calculated as Equation A 2.

$$EF = BER \times AADT \times CU \times GD \quad A 2$$

where

EF = encroachment frequency (encroachment/year/km)

BER = base encroachment rate (0.00030 encroachments/km/year/vpd)

CU = curvature adjustment factor

GD = grade adjustment factor

The base encroachment rate of 0.00030 is taken from AASHTO (2011). This rate should be adjusted when actual data at a location is available or modified based on engineering adjustment for non-typical conditions.

4. Determine severity index (SI) for roadside object

The SI defines the severity of the outcome of an impact with a particular roadside feature.

According to Austroads (2020), the severity indices are based on average crash costs when a vehicle impacts the hazard. The SI can be derived from AASHTO (2011) for traffic barriers; however, these values which are valid for occupants of light vehicles, are not suitable for motorcyclists and are probably not appropriate for trucks. If the situation has more than one factor, the case with the highest SI should be selected. According to Queensland Department of Transport and Main Roads (2005), however, a separate SI for each impact zone of the hazard should be applied.

5. Determine crash costs

The estimated crash costs per year can be calculated as:

$$\text{Annual crash costs per year (\$)} = (\text{impacts per year}) \times (\text{severity index crash cost per impact})$$

Based on the calculated SI, the severity index crash cost per impact is derived from a table issued by the relevant jurisdiction. In this table the costs related to different SI values ranging from 0 to 10 are provided. The percentage values and the associated costs of the crash outcomes (property damage, minor injury, moderate injury, hospitalisation and fatal) may all vary between jurisdictions.

These crash costs are based on Road Crash Costs in Australia (Bureau of Transport Economics 2000). Current values as determined by the Australian Bureau of Statistics should be used consistently throughout the jurisdictions.

Comparing different performance levels, the primary benefit obtained from selecting one design over another is the expected reduction in the future crash costs.

6. Determine barrier cost

The barrier costs include the cost of the barrier (per m), barrier maintenance cost (per m per year) and barrier repair cost (per penetration per year). While the cost of the barrier is the present value, the other 2 costs are assumed to be equally distributed over the design life (DL). Their associated present values should be calculated.

7. Determine BCR

The BCR between 2 bridge barrier alternatives is calculated by Equation A 3

$$\text{BCR} = (\text{NB1} - \text{NB2}) / (\text{NC2} - \text{NC1}) \quad \text{A 3}$$

where

NB1 = net present values of benefit of barrier alternative 1
(e.g. Medium performance level barrier)

NB2 = net present values of benefit of barrier alternative 2
(e.g. Special performance level barrier)

NC1 = net present value of cost of barrier alternative 1

NC2 = net present value of cost of barrier alternative 2.

Appendix D Bridge Support Protection Barrier

The supports (abutments and piers) of overpass bridges are required to be structurally protected from vehicle collisions. They are considered roadside hazards and therefore need to have safety treatments to protect road users.

It is specified in Section 15 of AS 5100.1:2017 that jurisdictions need to assess the risk of a vehicle impacting bridge and/or adjacent structures and determine the level and form of protection. A detailed procedure for risk assessment and determination of the protection level, however, is not available. This section provides a design guideline for bridge support protection barriers considering the following:

- design requirements
- determination of level of protection
- barrier layout and design.

D.1 Additional requirements to AS 5100 from jurisdictions' guidelines

All jurisdictions follow the AS 5100.1:2017 requirements for the design of bridge support protection barriers. Several jurisdictions such as Transport for NSW (2023) and the Victorian Department of Transport and Planning (2023) provide clarifications and more detailed guidelines.

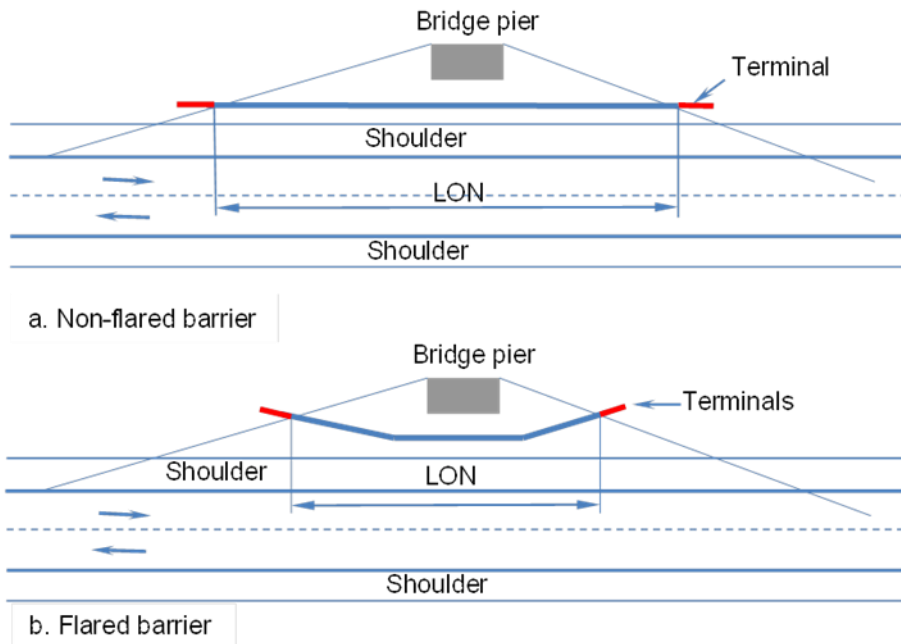
D.2 Barrier layout and design

D.2.1 Typical barrier layout for bridge supports adjacent to undivided roadways

For bridge supports located adjacent to undivided roadways, typical layouts are shown in Figure D.1. Non-flared and flared barriers may be selected based on the batter slope and available clearance. Single-sided barrier cross-sections can be used.

It is noted in Section 6.8.8 of Austroads (2024) that when a barrier or terminal is flared, the effect is to increase the likely impact angles by the flare angle. Caution should be used in applying flaring as barriers are designed to work best with a glancing impact. Flaring may lead to vehicles impacting a barrier at a high angle that will lead to hazardous consequences. Accordingly, the longer flare rates (and small flare angles) should be used when the barrier is installed closer to the traffic lane. Similarly, the maximum flare rates for concrete barriers are higher than those for more flexible barriers.

Figure D.1: Typical barrier layouts for bridge supports adjacent to undivided roadways

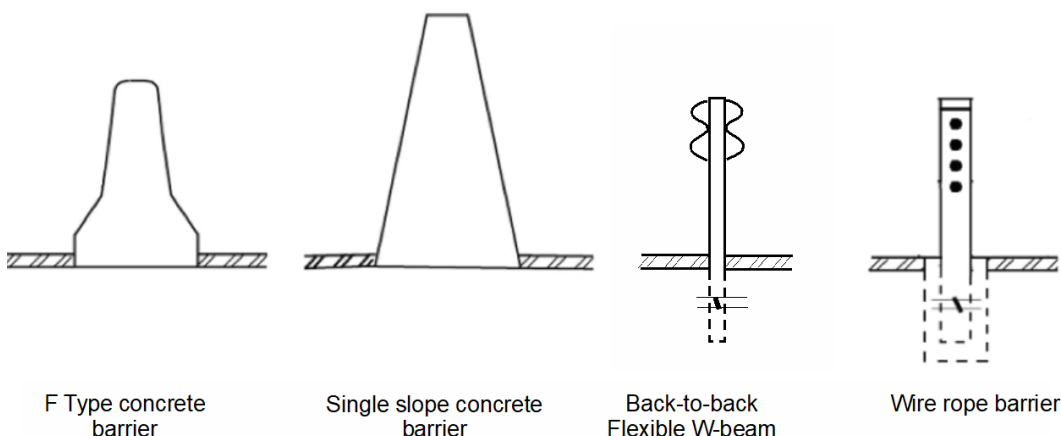


D.2.2 Typical barrier layout for bridge supports located in median area

Single-stage barrier system

For bridge piers located within a narrow median, F-shape or single slope rigid barriers are often chosen, but back-to-back semi-rigid or flexible barriers may be used where the dynamic deflection can be contained within the median width. Typical profiles of median barriers for use in narrow medians are shown in Figure D.2. Appropriate end treatments must be used to suit each type of barrier and situation (i.e. width available and behaviour of end treatment on vehicle impact).

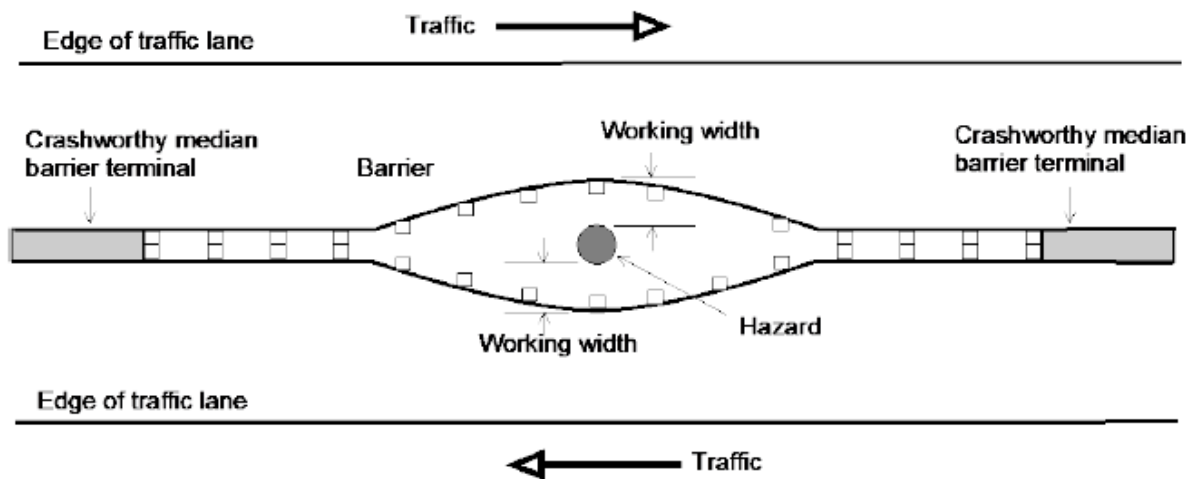
Figure D.2: Examples of road safety barriers for use in narrow medians



Source: Austroads (2024).

Figure D.3 shows how a bridge pier can be shielded in a narrow median. Where the median is wide enough to accommodate the barrier's deflection, a flexible or semi-rigid barrier is preferred.

Figure D.3: An example of a barrier layout for shielding a bridge pier located in a median

**Notes:**

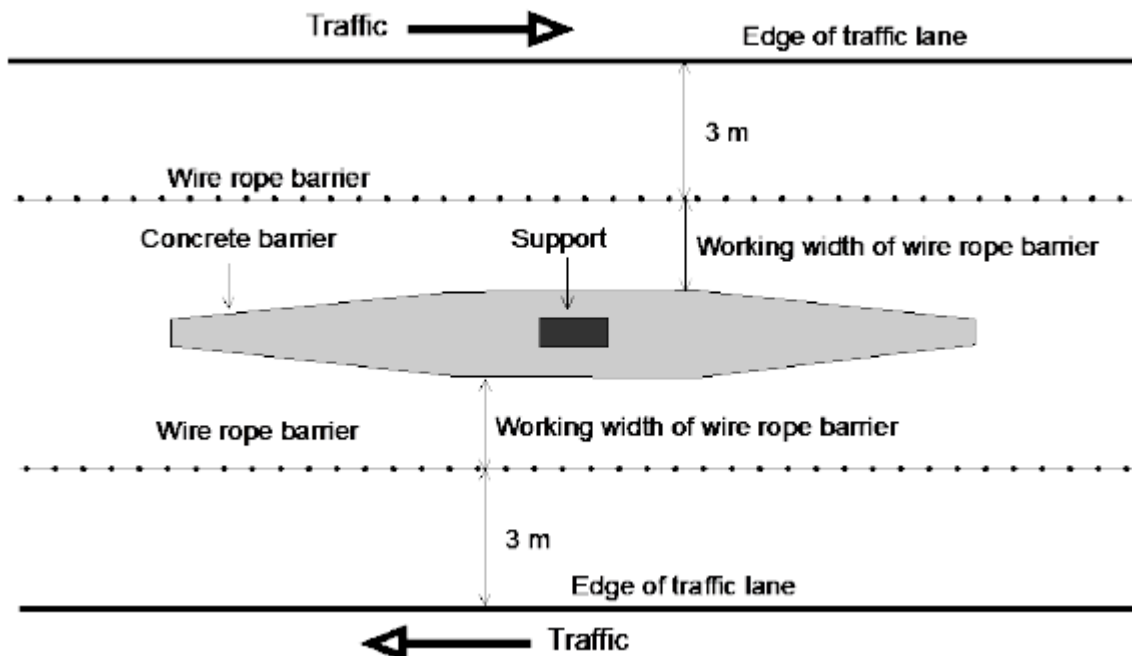
1. Designers should also investigate the use of a crash attenuator to shield the hazard.
2. The barrier tapers should be aligned to be consistent with the flare rates, as suggested in Section 6.8.8.

Source: Austroads (2024).

Two-stage barrier system

For bridge supports located in a wider median, a two-stage protection barrier system may be considered (Figure D.4).

Figure D.4: Two-stage protection layout

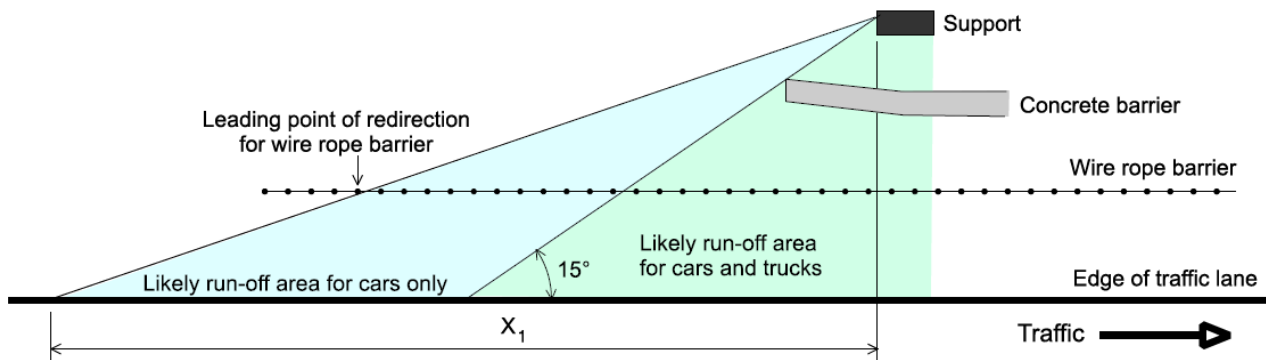


Source: Victorian Department of Transport and Planning (2019)

The following issues must be considered when designing two-stage protection (Austroads 2024):

- The 'run-out-length method' must be applied to the primary (front) barrier intended to protect passenger car occupants. This is the minimum length of barrier to address the run-off-road risk. The secondary (back) barrier is to prevent heavy vehicles from colliding with the gantry or support. The leading point of need for the concrete barrier is based on a 15° departure angle for heavy vehicles. Refer to Figure D.5.
- The traffic face of the rigid barrier must be offset from the face of the bridge pier to accommodate working width.
- The structural design of the bridge pier may require a gap between the concrete barrier and the bridge pier. Alternatively, the rigid barrier may be an integral part of the bridge pier, if the pier has been designed for high-mass, high-speed impact loadings.
- The rigid barrier must be designed to ensure the vehicle does not overturn it when impacted by errant large mass vehicles.
- If it is likely that the rigid barrier will be extended when additional lanes are added in the future, it is preferable that the rigid barrier be aligned parallel to the carriageway.
- In some situations, the end of the rigid barrier may require a crash attenuator if it is likely that passenger cars will be able to impact the end of the barrier.
- A cross slope of 10% or flatter should be provided between the flexible barrier and the concrete barrier.

Figure D.5: Points of need for two-stage shielding



Note: 1. X_1 is calculated using the length of need calculation in Section 6.9.1 of AGRD Part 6 (Austroads 2024).
2. The design departure angle for trucks is assumed to be 15°.

Source: Austroads (2024).

D.2.3 Height considerations

The height of the road safety barriers shall be in accordance with Austroads (2024).

The height of the bridge support protection barrier associated with the barrier performance level may be modified to safely redirect a heavy commercial vehicle and minimise the lateral rotation of a high centre-of-gravity vehicle and potential for snagging of the bridge supports.

The determination of appropriate height and clearance of the pier protection system from the bridge supports shall be based on the barrier working width (Section 7.3.1), which includes consideration for vehicle roll allowance (Figure 7.1 and Table 5.1). For rigid concrete barriers, the minimum effective height of the barrier shall be in accordance with the performance level selected for the site as per Section 3. Minimum effective heights are provided in Table 5.1.

The height of the protection barrier should be gradually reduced away from the bridge supports to reduce the size of the potential impact area at each end of the system. However, the slope on the top surface of the barrier should minimise the probability of the barrier acting as a ramp and launching a heavy vehicle. This slope should be of the order of 1 in 10. Appropriate end treatments must be used to suit each type of barrier and situation (i.e. width available and behaviour of end treatment on vehicle impact).

For applications where the operational speed of the road is 80 km/h or less, site-specific risk assessment might indicate that a reduced height of about 1,100 mm might be adequate to re-direct heavy vehicles, subject to considerations about the probability of bridge support snagging and the robustness of these supports.

D.2.4 Foundations for rigid barriers

Where rigid barriers are provided, appropriate foundations, in the form of bored piles or similar, shall be provided to resist the barrier design loads referred to above in accordance with the requirements of AS 5100:2017.

D.3 Bridge support protection from railway traffic

AS 5100:2017 provides the following considerations for the design of bridges and other structures over or adjacent to railways:

- Unless approved otherwise by the relevant authorities, structures over rail shall have a clear span between abutments.
- Where the relevant authorities approve piers between abutments, and the nearest face of the pier is closer than 10 m from the centreline of existing or proposed tracks, provision for impact from a derailed train shall be in accordance with Clauses 15.3.3 to 15.3.9 of AS 5100.1:2017.
- Where the nearest pier face is between 10 m and 20 m from the centreline of existing or proposed tracks, a risk assessment, to the approval of the relevant authorities, shall be carried out to determine the requirements for protection of supports and the loading shall be in accordance with Clause 11.4.2.4 of AS 5100.2:2017.
- For road or pedestrian bridges over rail, where the nearest face of the pier is within 10 m from the centreline of existing or proposed tracks, the pier shall be a frangible pier in accordance with Clause 15.3.3 of AS 5100.1:2017, or if a frangible pier is not reasonably practicable then the pier shall meet the requirements of Clause 15.3.4 of AS 5100.1:2017.
- Rail bridges shall not be supported on frangible piers.
- A risk assessment that includes the risks associated with the use of any pier type and superstructure articulation shall be undertaken. The risk assessment methodology shall be acceptable to all relevant authorities.

Bridge supports that are designed for the minimum collision load given in AS 5100.2:2017, can only be expected to deflect a train when the collision is a glancing blow, not a head-on collision of a locomotive. In a head-on collision, the pier, column or wall may fail, resulting in support failure, or the train may telescope, concertina or become partly airborne, in which case the support and/or superstructure may be impacted and fail.

Continuous walls are required in order to deflect a derailed train and prevent the demolition of bridge piers or structure supports.

Deflection walls significantly improve the performance of supports of heavy construction in a derailment. They perform more effectively the more they can deflect in a derailment before loading the support.

The wall geometry should be selected so that a derailed locomotive is maintained in an upright position if possible.

In some circumstances, it may be impractical to design for the loads specified in AS 5100.2:2017 and the relevant authority may specify an alternative load condition.

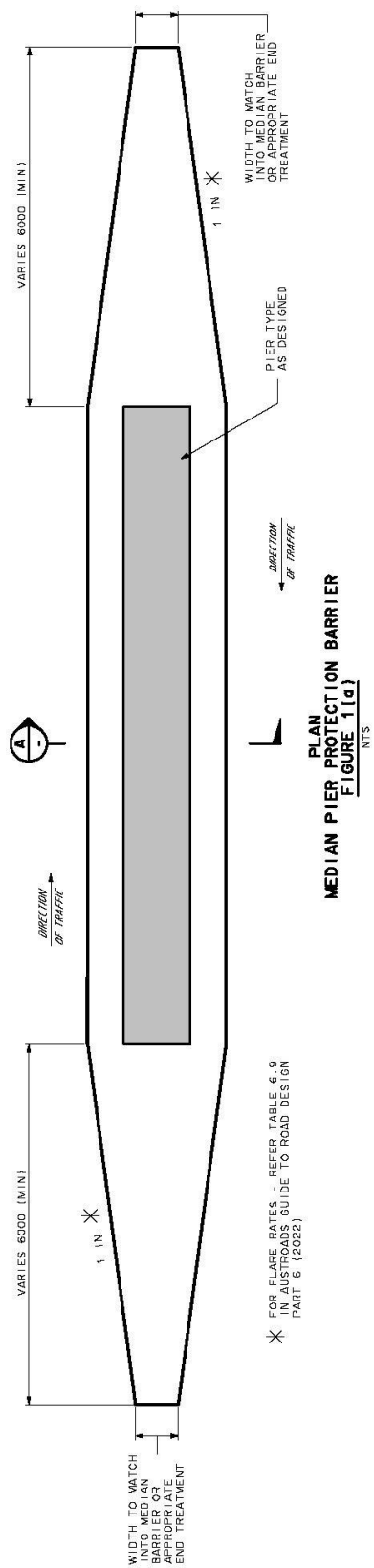
Reference should be made to Queensland Rail (2011) for a comprehensive guidance on collision protection of supporting elements adjacent to railways.

D.4 Typical design drawings

This section presents concept details of typical systems currently being used by the Victorian Department of Transport and Planning for the collision protection of existing and new bridge supports located in the central median of freeways. Similar details can be used on major highways and arterial roads with divided carriageways.

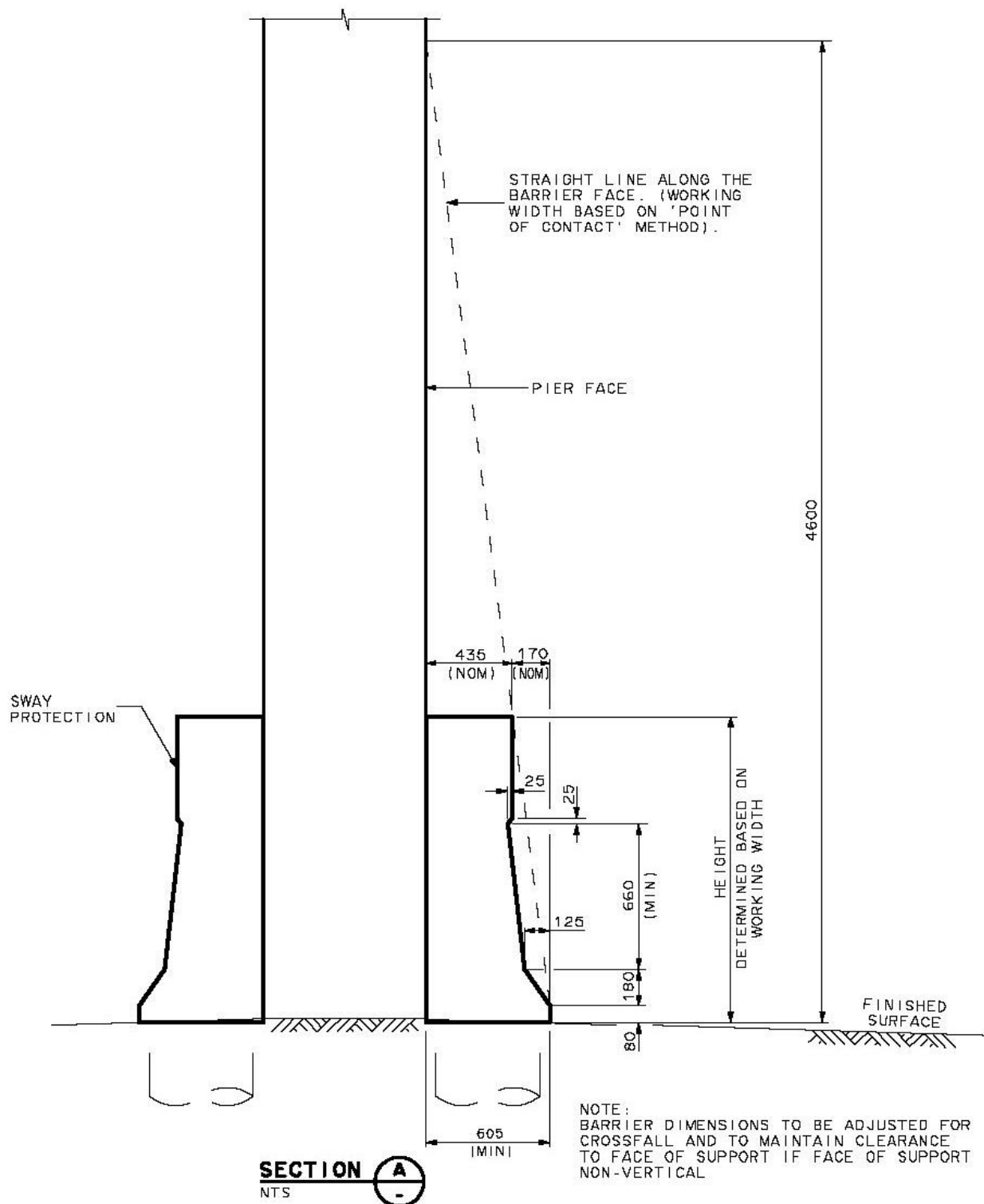
Plan view and side elevation for a typical median pier collision protection system are shown in Figure D.6. The typical cross-sections are shown in Figure D.7 **Error! Reference source not found.**, Figure D.8 and Figure D.9. These cross-sections contain the 'sway protection modifications' proposed by the Victorian Department of Transport and Planning (DTP) to reduce body roll. The modification still allows for vehicles to be smoothly redirected. However, its use should be a design exception with relevant jurisdictional approval (Austroads 2024). DTP does acknowledge that sway modifications can reduce vehicle roll for trucks when hazards cannot be relocated or moved further back. It is critical that the balance between reducing roll and affecting passenger vehicle impacts is managed. When a typical passenger vehicle impacts the F-shape, it mounts the kerb reveal and slides up the barrier before being redirected. Modifications to the upper profile may change the severity of the impact and should be avoided (Victorian Department of Transport and Planning 2021b).

Figure D.6: Example: VicRoads typical bridge support protection system



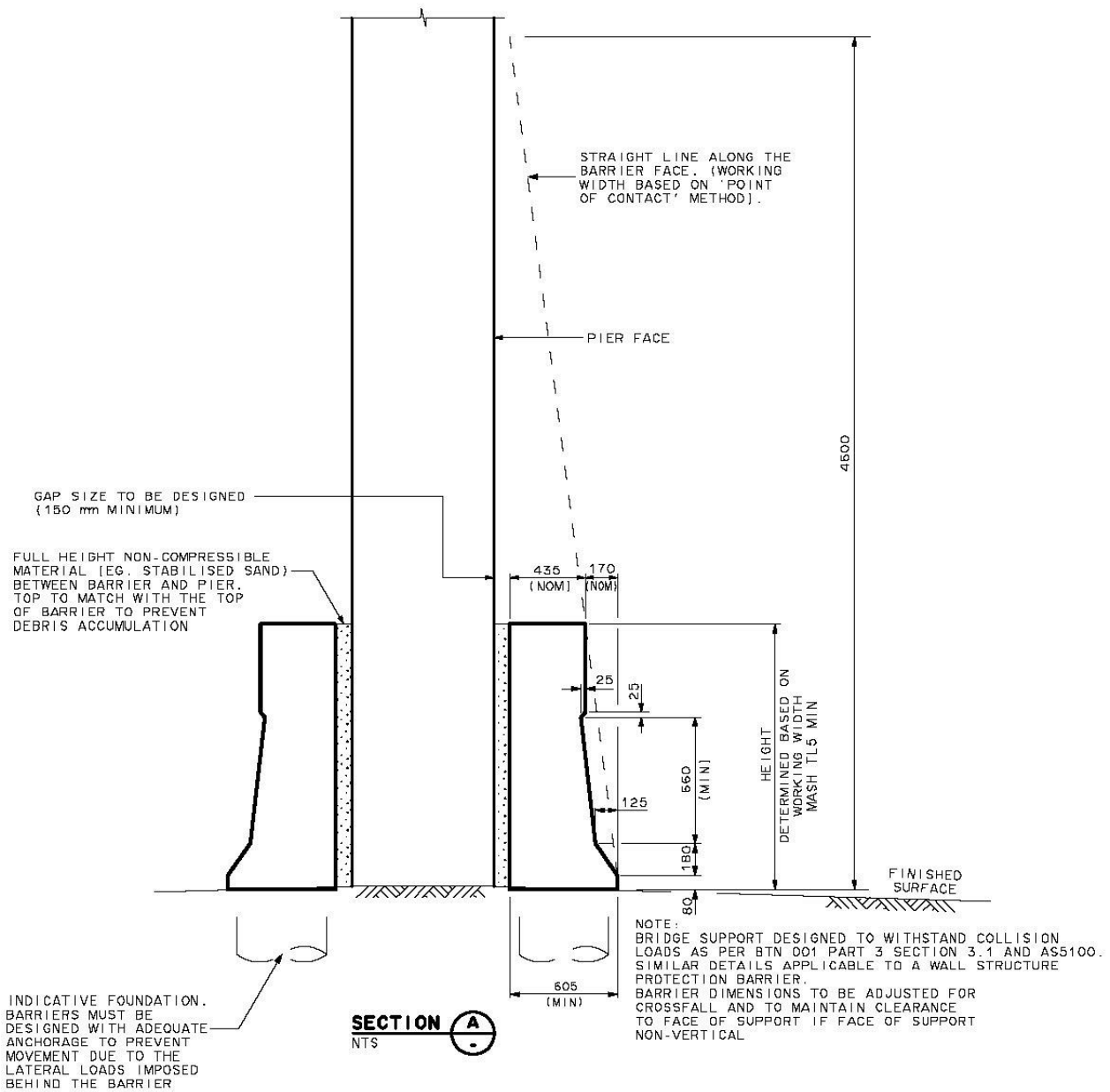
Source: Victorian Department of Transport and Planning (2023).

Figure D.7: Typical collision protection system for a combined bridge support column and protection barrier



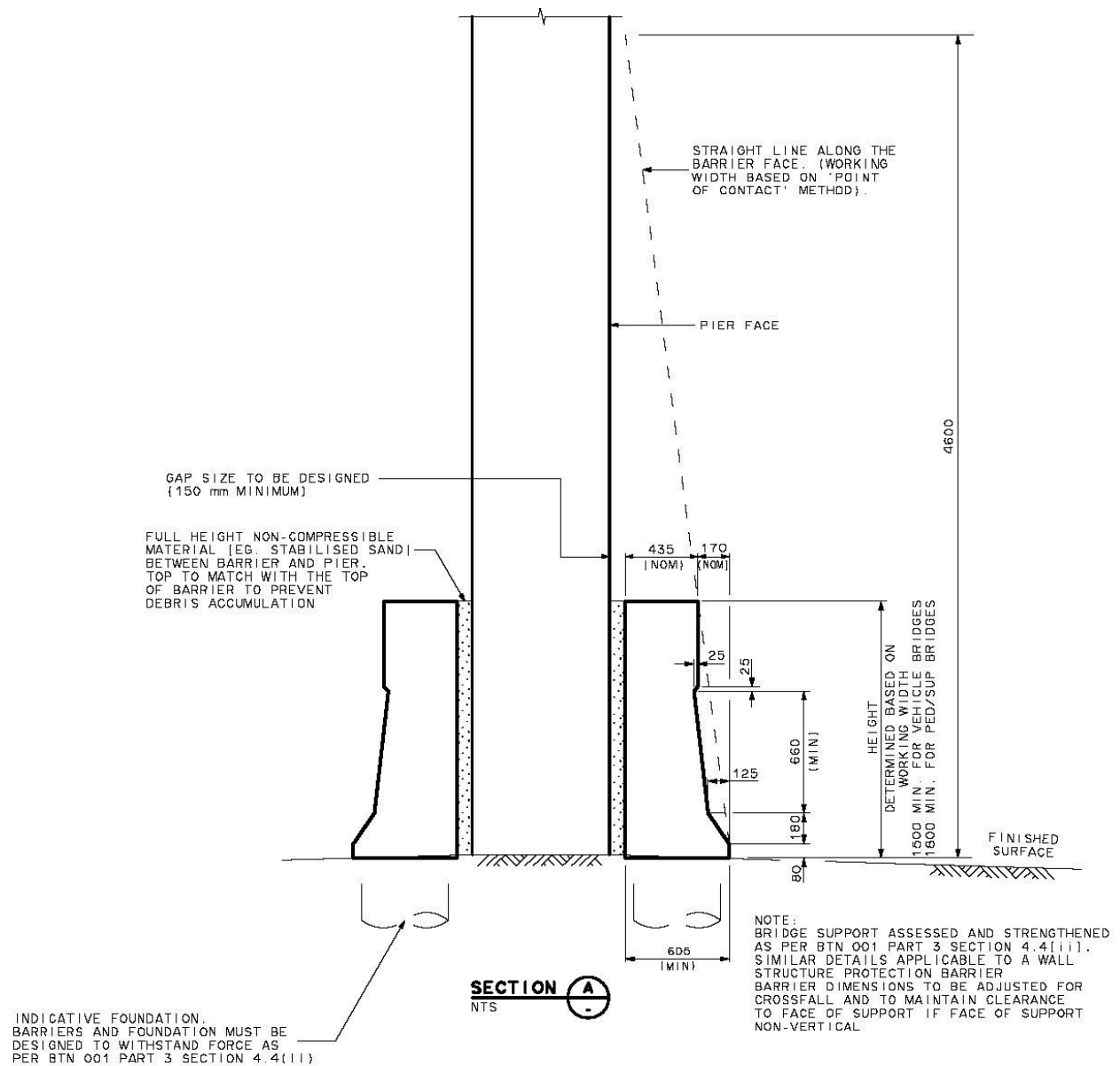
Source: Victorian Department of Transport and Planning (2023).

Figure D.8: Typical collision protection system for an independent bridge support designed for collision loads and protection barrier



Source: Victorian Department of Transport and Planning (2023).

Figure D.9: Typical collision protection system for an existing bridge support which cannot withstand collision loads and protection barrier

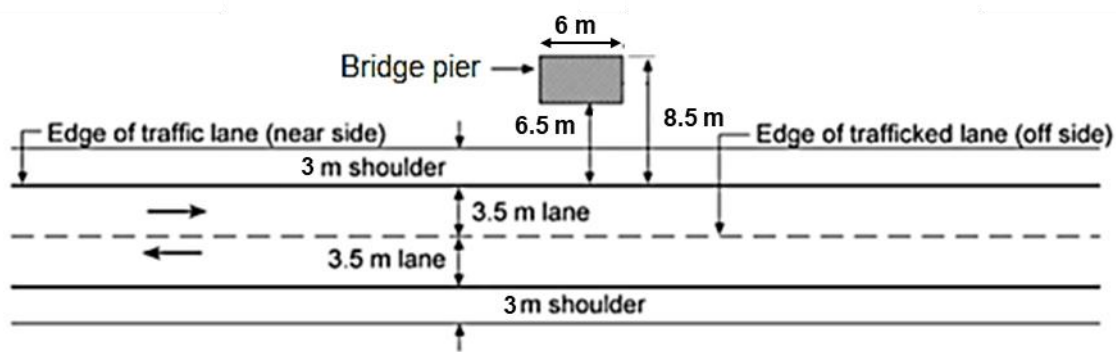


Source: Victorian Department of Transport and Planning (2023).

D.5 Design example 1: Protection barrier for a bridge pier adjacent to an undivided two-way road

Determine the layout of a road safety barrier to shield a bridge pier (6 m long x 2 m wide) adjacent to a rural two-lane, two-way road as illustrated in Figure D.10.

Figure D.10: Example 1: Bridge pier protection barrier



The road has the following characteristics:

AADT = 14,000

Commercial vehicles = 2%

Design speed = 100 km/h

Alignment: Essentially straight with flat gradient

Plan view: As per Figure D.10; 3.5 m shoulder; pier 6.5 m from the edge of the traffic lane; fill batter slope 6:1.

D.5.1 Determine level of protection

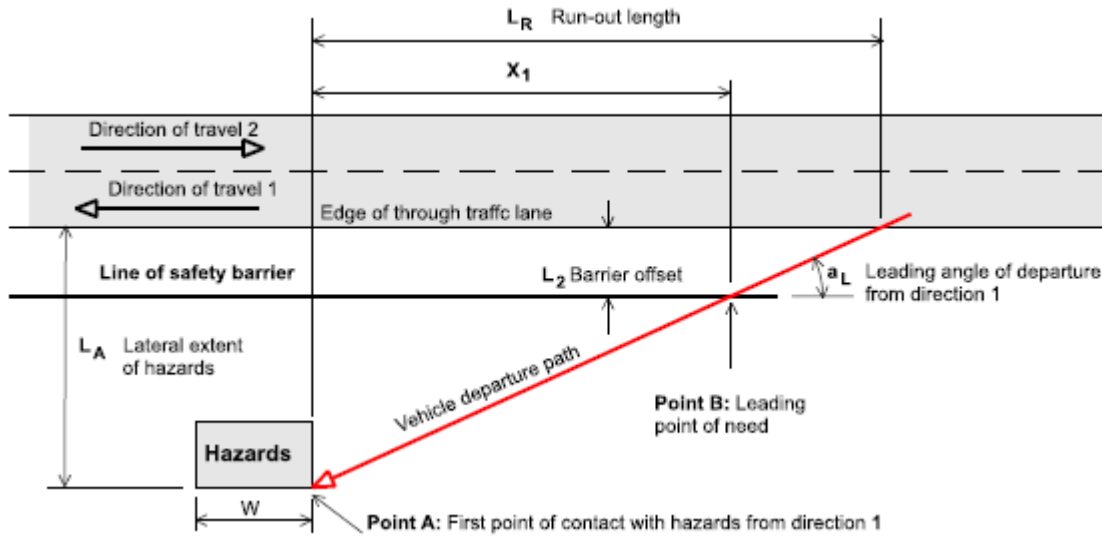
Section 6.20.2 of the *Guide to Road Design Part 6: Roadside Design and Safety Barriers* (Austroads 2024) requires that barriers shielding bridge piers are TL-5 concrete barriers. There is a preference for barriers that are 1,370 mm high and have an F-shape or single slope profile. The working width for this type of barrier is 1.5 m. It is appropriate to apply a factor of safety to the working width of barriers protecting critical infrastructure. The factor of safety is recommended to be 1.2 (Austroads 2024).

D.5.2 Length of need

The length of need is the length of barrier required to redirect an errant vehicle and shield hazards and is defined by the distance between the leading and trailing points of need (Austroads 2024). The leading and trailing points of need are calculated in this section (Figure D.11 to Figure D.15).

Leading point of need

Figure D.11: Run-out length method to determine the leading point of need on two-way roads



Source: Austroads (2024).

Consider first the lane nearest the hazard. The leading point of need for traffic is calculated by considering the lane nearest to the hazard on the approach to the hazard, where:

L_R = Run-out length, as per Table G.1 in *Austroads Guide to Road Design: Part 6* (Austroads 2024) = 91 m

L_A = Lateral extent of the hazard = 8.5 m

L_2 = Barrier's lateral distance from the edge of the traffic lane. Assume that the barrier is located at a 0.3 m lateral distance from the edge of the shoulder, then
 $L_2 = 3.0 \text{ m} + 0.3 \text{ m} = 3.3 \text{ m}$

X_1 = The location of the leading point of need in advance of the hazard

$$X_1 = \max \left\{ \left(\frac{L_R}{L_A} \right) (L_A - L_2), 8(L_A - L_2) \right\}$$

$$X_1 = 56 \text{ m}$$

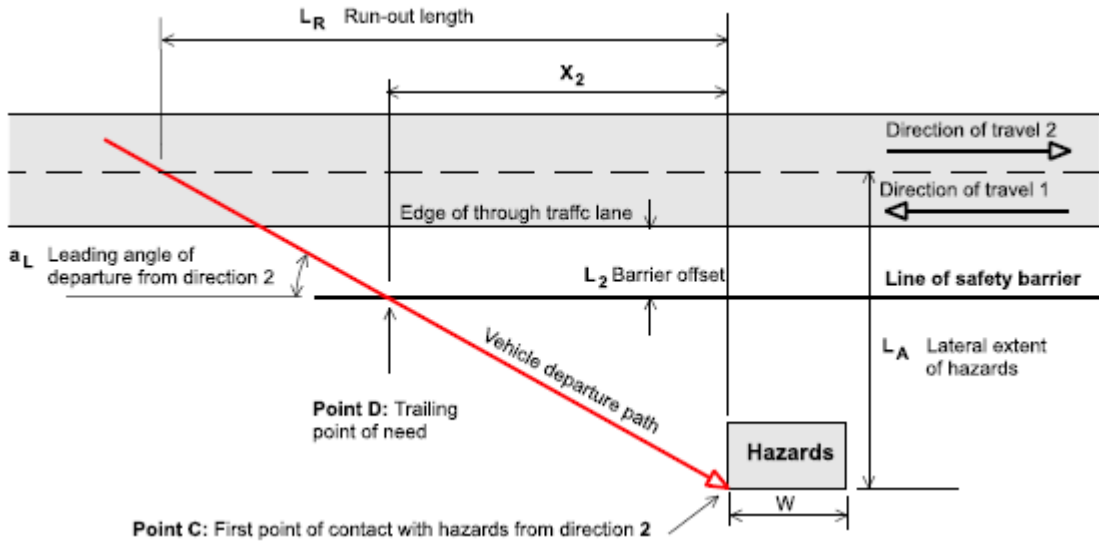
Therefore, $X_1 = 60 \text{ m}$

Note that this is the distance from the leading face of the pier to the point of need for traffic approaching in the lane nearest the hazard.

The end of the wire rope barrier should be shielded using a suitable leading terminal which will increase the overall length.

Trailing point of need

Figure D.12: Trailing point of need for roads with two-way traffic



Source: Austroads (2024).

L_R = Run-out length, as per Table G.1 in Austroads *Guide to Road Design: Part 6* (Austroads 2024) = 91 m

L_A = Lateral extent of the hazard = 12 m

L_2 = Barrier's lateral distance from the edge of the traffic lane. Assume that the barrier is located at a 0.3 m lateral distance from the edge of the shoulder, then
 $L_2 = 3.0 \text{ m} + 0.3 \text{ m} = 3.3 \text{ m}$

X_2 = The location of the leading point of need in advance of the hazard

$$X_2 = \max \left\{ \left(\frac{L_R}{L_A} \right) (L_A - L_2), 8(L_A - L_2) \right\}$$

$$X_2 = 70 \text{ m}$$

Therefore, $X_2 = 70 \text{ m}$

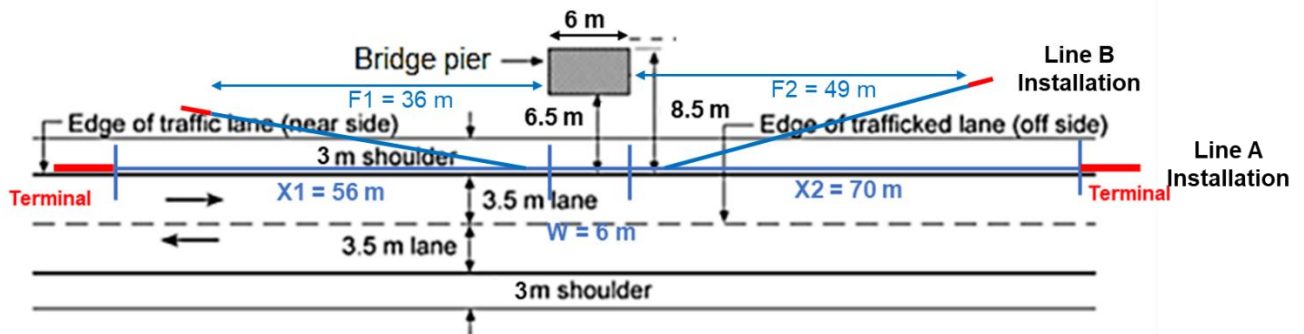
Note that this is the distance from the trailing face of the pier to the point of need for traffic approaching in the lane furthest from the hazard. A suitable trailing terminal which will increase the overall length should also be provided.

The overall length of road safety barrier required for traffic approaching from both directions therefore comprises the sum of the length required for the traffic in the lane adjacent to the hazard (leading), the length required for traffic in the opposing lane (trailing), plus the length of pier parallel to the road (i.e. 6.0 m).

Hence, the overall length of need of the barrier is $56 + 70 + 6.0 = 132 \text{ m}$.

The road safety barrier layout determined above is illustrated as the Line A installation (straight barrier) in Figure D.13.

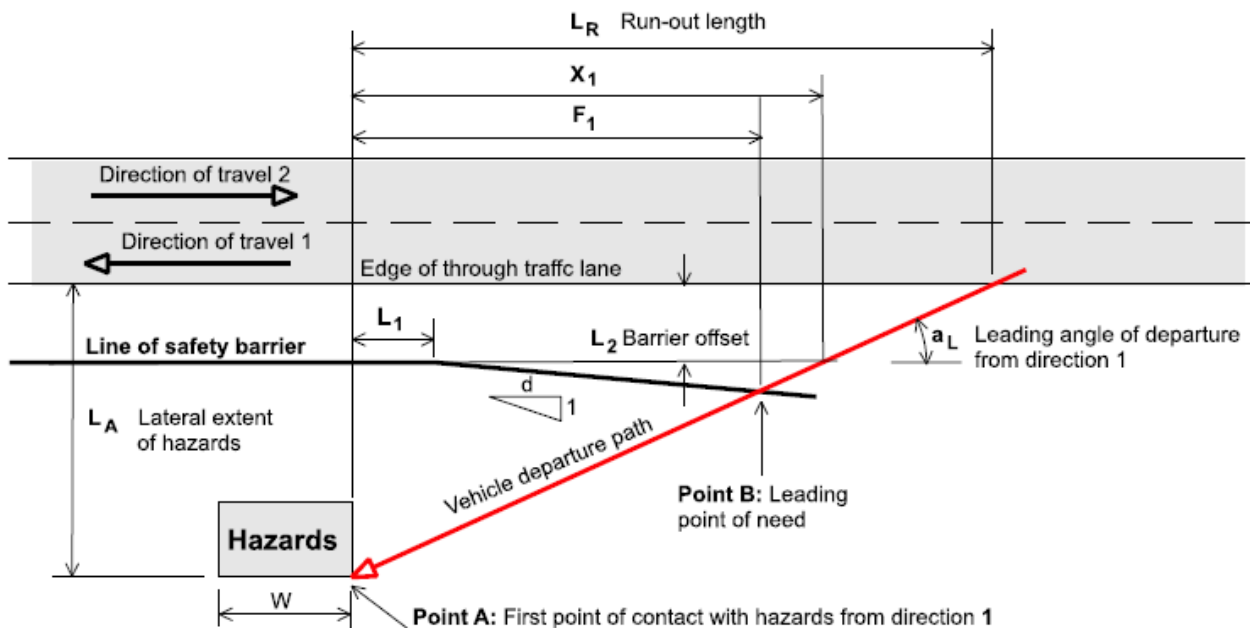
Figure D.13: Example 1: Options for bridge pier protection barrier layout



A Line B installation is also illustrated as a flared installation. This requires a shorter road safety barrier. The effects, advantages and disadvantages of flaring barriers and terminals are discussed in Section 6.8.8 of Austroads (2024).

Leading point of need for flared installation

Figure D.14: The leading point of need for a flared installation



Source: Austroads (2024).

Consider first the lane nearest the hazard. The leading point of need for traffic is calculated by considering the lane nearest to the hazard on the approach to the hazard, where:

L_R = Run-out length, as per Table G.1 in Austroads *Guide to Road Design Part 6* (Austroads 2024) = 91 m

L_A = Lateral extent of the hazard = 8.5 m

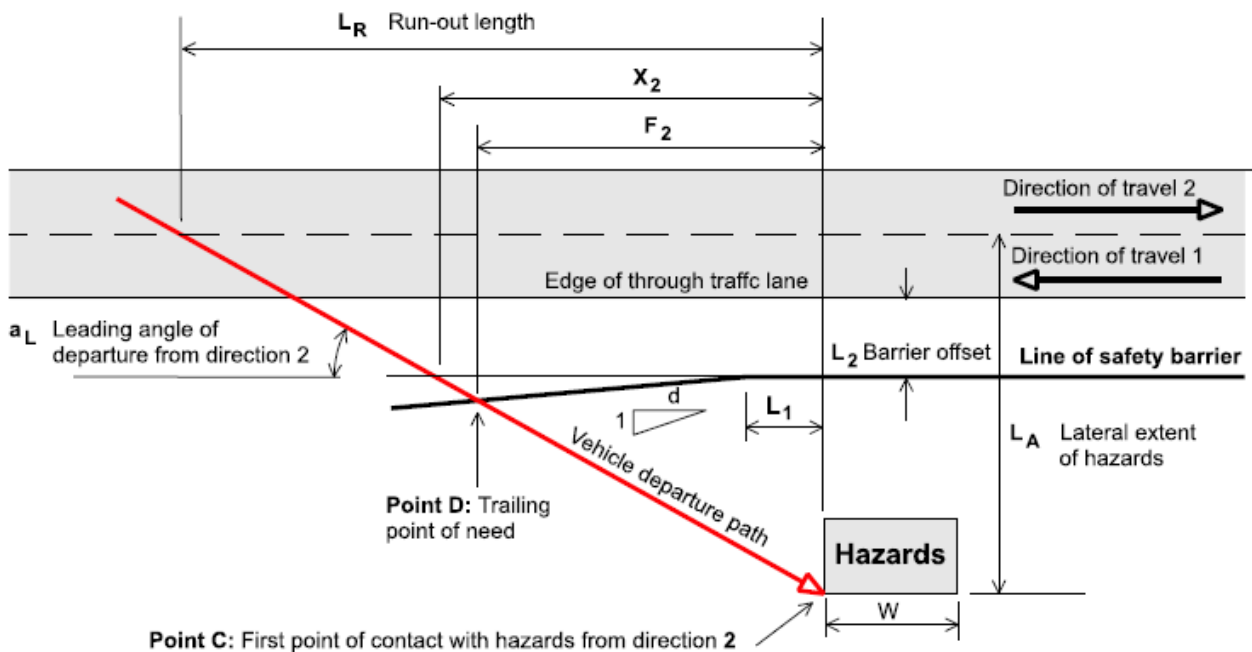
- L_2 = Barrier's lateral distance from the edge of the traffic lane. Assume that the barrier is located at a 0.3 m lateral distance from the edge of the shoulder, then
 $L_2 = 3.0 \text{ m} + 0.3 \text{ m} = 3.3 \text{ m}$
- L_1 = Distance of the start of the flare from the hazard. In this case, L_1 can be taken as 3.0 m
- $d:1$ = Barrier flare rate. Using Table 6.9 in Austroads Guide to Road Design Part 6 (Austroads 2024), for an operating speed of 100 km/h and the flare rate for stiffer barriers further from the road (barrier offset greater than 3.0 m) the flare rate can be taken as 18:1
- F_1 = The location of the leading point of need in advance of the hazard

$$F_1 = \max \left\{ \left(\frac{L_A - L_2 + \frac{L_1}{d}}{\frac{1}{d} + \frac{L_A}{L_R}} \right), \left(\frac{L_A - L_2 + \frac{L_1}{d}}{\frac{1}{d} + \frac{1}{8}} \right) \right\}$$

$$F_1 = 36 \text{ m}$$

Trailing point of need for flared installation

Figure D.15: The trailing point of need for a flared installation on a road with bidirectional traffic



Source: Austroads (2024).

Consider first the lane nearest the hazard. The leading point of need for traffic is calculated by considering the lane nearest to the hazard on the approach to the hazard, where:

- L_R = Run-out length, as per Table G.1 in *Austrroads Guide to Road Design Part 6* (Austrroads 2024) = 91 m
- L_A = Lateral extent of the hazard = 12 m
- L_2 = Barrier's lateral distance from the edge of the traffic lane. Assume that the barrier is located at a 0.3 m lateral distance from the edge of the shoulder, then
 $L_2 = 3.0 \text{ m} + 0.3 \text{ m} = 3.3 \text{ m}$
- L_1 = Distance of the start of the flare from the hazard. In this case, L_1 can be taken as 3.0 m
- d:1 = Barrier flare rate. Using Table 6.9 in *Austrroads Guide to Road Design Part 6* (Austrroads 2024), for an operating speed of 100 km/h and the flare rate for stiffer barriers further from the road (barrier offset greater than 3.0 m) the flare rate can be taken as 18:1
- F_2 = The location of the trailing point of need

$$F_2 = \max \left\{ \left(\frac{L_A - L_2 + \frac{L_1}{d}}{\frac{1}{d} + \frac{L_A}{L_R}} \right), \left(\frac{L_A - L_2 + \frac{L_1}{d}}{\frac{1}{d} + \frac{1}{8}} \right) \right\}$$

$$F_2 = 49 \text{ m}$$

Hence, the overall length of need of the barrier is $36 + 49 + 6.0 = 91 \text{ m}$, which is 41 m shorter than the LoN for the straight installation.

D.6 Design Example 2: Protection Barrier for a Bridge Pier Located in a Median

Determine the layout of a barrier to shield the end of a bridge barrier on a divided rural highway as illustrated in Figure D.16. The road at the bridge approach section has the following data:

AADT: 30,000 vpd

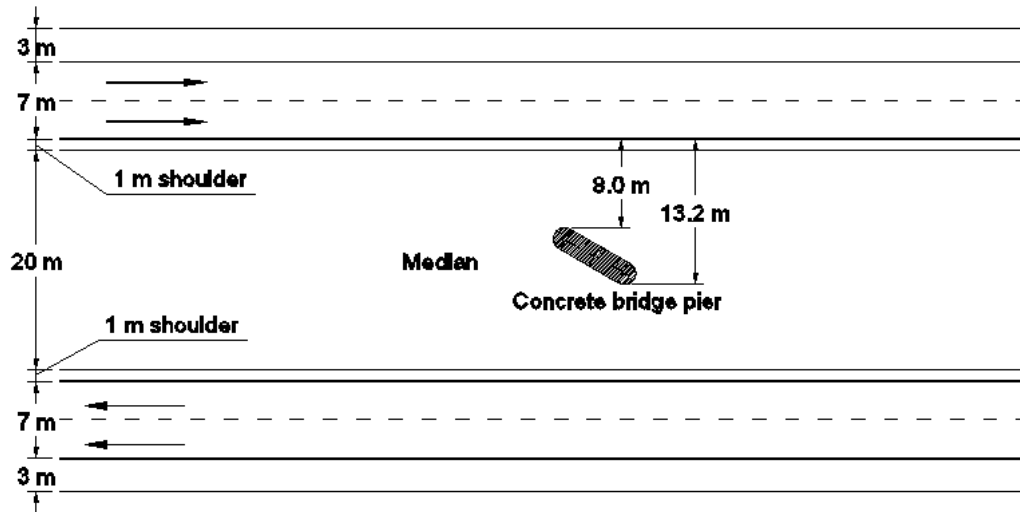
Commercial vehicles = 20%

Design speed: 100 km/h

Alignment: Essentially straight with flat gradient

Plan: A concrete bridge pier located in the median area of a two-way divided road; 3.0 m shoulder on left, 1 m shoulder on right; batter slope in the median 6:1.

Figure D.16: Example 2: Site plan



D.6.1 Determine level of protection

The same assessment as in Example 1 (Section D.5) was carried out for this example. The road barrier performance level was selected in line with Appendix A of AS 5100.1:2017. The performance level was determined using site conditions including design speed, percentage of commercial vehicles and adjusted AADT. With a design speed of 100 km/h, an adjusted AADT of 30,000 vehicles/day and commercial vehicles making up 20% of total vehicles, Figure A7 in AS 5100.1:2017 was used to select a Medium Performance Level TL-5 Barrier. TL-5 barrier systems are generally reinforced concrete or slip formed barriers with an F-shape. A reinforced concrete barrier system was selected for this design. The preliminary design working width (minimum distance between the face of the barrier and hazard) for the reinforced concrete system with TL-5 is 2.4 m for a barrier height of 1,070 mm according to Austroads (2024).

D.6.2 Length of need

The leading and trailing points of need are calculated as follows.

Leading point of need

Consider first the lane nearest the hazard. The leading point of need for traffic is calculated by considering the lane nearest to the hazard on the approach to the hazard, where:

L_R = Run-out length (Table G.1, Austroads Guide to Road Design Part 6 [2024]) = 91 m

L_A = Lateral extent of the hazard = 13.2 m

L_2 = Barrier's lateral distance from the edge of the traffic lane. Assume that the barrier is located at a 0.6 m lateral distance from the edge of the shoulder, then
 $L_2 = 1.0 \text{ m} + 0.6 \text{ m} = 1.6 \text{ m}$

X_1 = The location of the leading point of need in advance of the hazard

$$X_1 = \max \left\{ \left(\frac{L_R}{L_A} \right) (L_A - L_2), 8(L_A - L_2) \right\}$$

$$X_1 = 93 \text{ m}$$

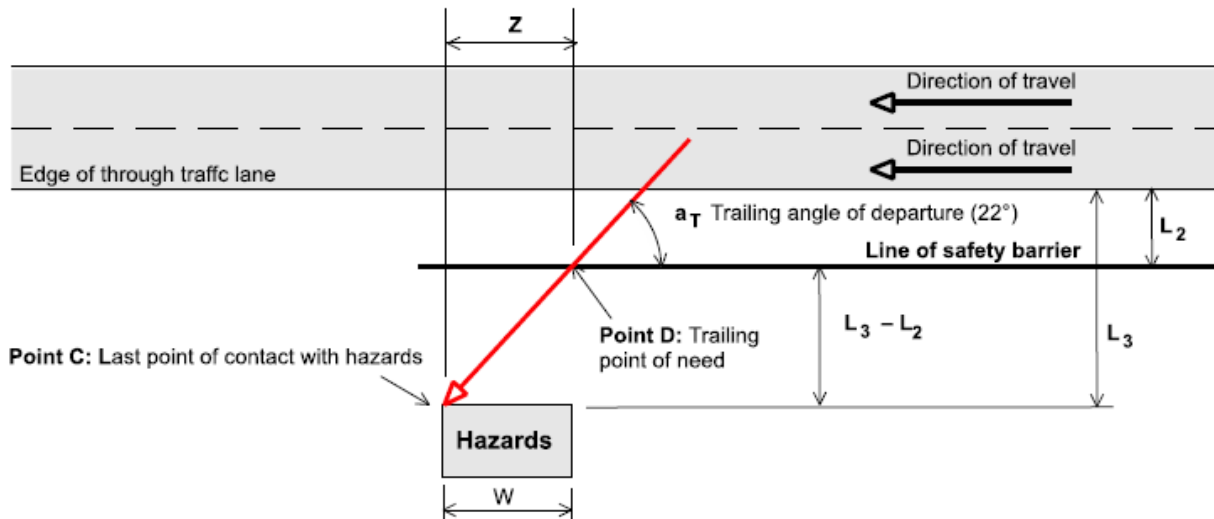
Therefore, $X_1 = 93$ m

Note that this is the distance from the leading face of the pier to the point of need for traffic approaching in the lane nearest the hazard. The end of the concrete barrier should be shielded using a suitable leading terminal (i.e. crash cushion).

Trailing point of need

For one-way carriageways, the trailing point of need is illustrated in Figure D.17.

Figure D.17: Trailing point of need on a one-way carriageway



Source: Austroads (2024).

- L_R = Run-out length (Table G.1, Austroads *Guide to Road Design Part 6* [Austroads 2024]) = 91 m
- L_3 = Distance from the edge of the traffic lane to the edge of the hazard closest to the road = 8.0 m
- L_2 = Barrier's lateral distance from the edge of the traffic lane. Assume that the barrier is located at a 0.6 m lateral distance from the edge of the shoulder, then $L_2 = 1.0 \text{ m} + 0.6 \text{ m} = 1.6 \text{ m}$
- Z = The trailing point of need

$$Z = 4.0[L_3 - L_2]$$

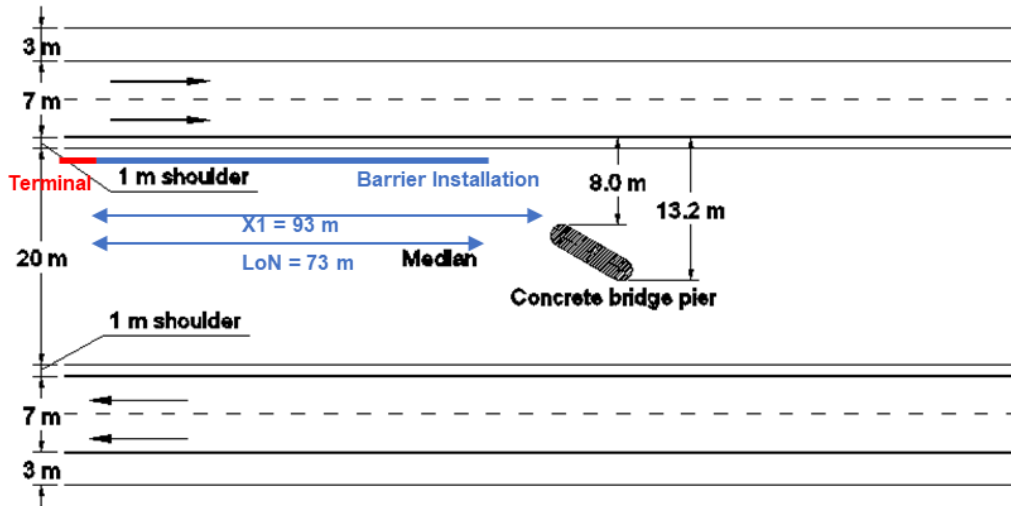
$$Z = 26 \text{ m}$$

The overall length of road safety barrier required for traffic approaching from both directions therefore comprises the sum of the length required for the traffic in the lane adjacent to the hazard, plus the length of pier parallel to the road (i.e. 6.0 m).

Hence the overall length of need of the barrier is $93 + 6.0 - 26 = 73$ m.

The road safety barrier layout determined above is illustrated as the barrier installation in Figure D.18.

Figure D.18: Straight barrier installation for design example 2



Consideration should be given to the distance from the barrier to the face of the pier to ensure the minimum working width is provided. A second layer of WRSB may be installed near the edge of traffic lanes in both directions to reduce the risk of errant vehicles colliding with the rigid barriers. The length of need can be reduced with the installation of a flared barrier. The effects, advantages and disadvantages of flaring barriers and terminals are discussed in Section 6.8.8 of Austroads (2024).

Leading point of need for flared installation

Consider first the lane nearest the hazard. The leading point of need for traffic is calculated by considering the lane nearest to the hazard on the approach to the hazard, where:

- L_R = Run-out length (Table G.1, Austroads Guide to Road Design Part 6 [Austroads 2024]) = 91 m
- L_A = Lateral extent of the hazard = 13.2 m
- L_2 = Barrier's lateral distance from the edge of the traffic lane. Assume that the barrier is located at a 0.6 m lateral distance from the edge of the shoulder, then
 $L_2 = 1.0 \text{ m} + 0.6 \text{ m} = 1.6 \text{ m}$
- L_1 = Distance of the start of the flare from the hazard. In this case, L_1 can be taken as 3.0 m
- $d:1$ = Barrier flare rate. Using Table 6.9 in Austroads Guide to Road Design Part 6 (Austroads 2024), with an operating speed of 100 km/h and the flare rate for barriers closer from the road (barrier offset less than 2.0 m from right hand side median) the flare rate can be taken as 26:1
- F_1 = The location of the leading point of need in advance of the hazards

$$F_1 = \max \left\{ \left(\frac{L_A - L_2 + \frac{L_1}{d}}{\frac{1}{d} + \frac{L_A}{L_R}} \right), \left(\frac{L_A - L_2 + \frac{L_1}{d}}{\frac{1}{d} + \frac{1}{8}} \right) \right\}$$

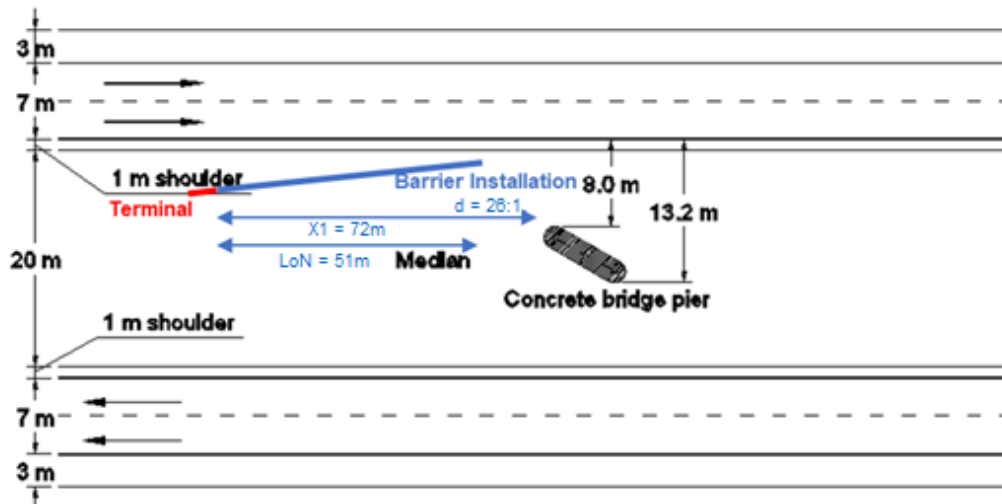
$$F_1 = 72 \text{ m}$$

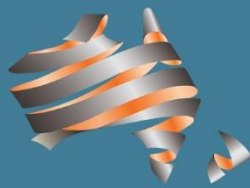
Trailing point of need for flared installation

As the road carriageways are unidirectional, flaring of the barrier does not affect the trailing point of need. Hence, the trailing point of need remains as 26 m as per the calculation above.

The overall length of need of the barrier is $71 + 6.0 - 26 = 51$ m, which is 22 m shorter than the LoN for the straight installation. See **Error! Reference source not found.**

Figure D.19: Flared barrier installation for design example 2





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