Design Guide
Weathering Steel

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Design Guide for Bridges in Australia

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Cover & Right Images: Bidgee Bidgee Bridge Rosehill NSW featuring REDCOR® weathering steel

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Foreword

Steel is used extensively for bridges around the world in various structural arrangements or systems. The advantage of steel as a material for bridges is its high strength-to-weight ratio. The importance of this increases with the length of the span as the lower dead load leads can lead to a more efficient structure and lower foundation costs. An additional advantage of steel bridges can be realised from the use of REDCOR® Weathering Steel. This steel is extremely durable and can result in lower whole of life costs compared with conventional carbon steel and concrete bridges, when used in the right environment and detailed correctly. These factors can collectively make RECCOR® weathering steel bridges a financially advantageous choice.

BlueScope produces a range flat and welded steel products made from REDCOR® weathering steel. Steel plate as well as welded beams and columns are typically used in the construction of a wide range of bridge types from short to long span road bridges, rail bridges and pedestrian bridges.

This guide has been prepared by engineers and materials experts from HERA in conjunction with BlueScope and our partners to assist Australian bridge designers with preliminary designs of economical REDCOR® weathering steel bridges. This edition is a comprehensive update to the original 2017 HERA Design Guide for Weathering Steel Bridges that considers the knowledge accumulated and new industry practices implemented since then. The aim of this guide is to support engineers' detail design of REDCOR® weathering steel bridges in Australia. It covers appropriate applications for weathering steel and provides guidance to achieve the expected performance and the realisation of design life of these bridges. Aspects of REDCOR® weathering steel covered in the guide include design, detailing, fabrication, construction, inspection, maintenance and rehabilitation, if it's required.

We have all put significant time and effort into developing this 2025 HERA Guide and we hope you find it a useful resource for your new bridge projects.

Rob Danis
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BlueScope Australian Steel Products

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1. Introduction

Weathering steel is a high strength, structural steel that, in suitable environments, develops a tightly adherent oxide layer or 'patina', which significantly reduces the corrosion rate compared with conventional structural steel.

Weathering steel has been used since the 1930's in railway coal wagons, bridges, buildings, facades and many architectural features such as sculptures and landscaping. It has been used extensively in North America, Europe and Japan for over 60 years; in Australia for nearly 50 years (see Figure 1-1 below) and nearly 40 years in New Zealand. When designed and detailed correctly, and considering the environmental factors that govern its use, it has exhibited excellent performance in reducing the corrosion rate compared with conventional structural steel.

A well designed and correctly detailed weathering steel bridge, in an appropriate environment, can provide an attractive, very low maintenance, economic solution and extends the scope for cost-effective steel bridges.

Accordingly, the Heavy Engineering and Research Association of NZ and Opus International Consultants (now WSP), NZ prepared the original version of this document entitled 'Weathering Steel Design Guide for Bridges in Australia' in 2017, which has now been updated to include new and some additional information. The purpose of this publication is to provide a collation of the necessary guidance for the Australian industry to assist with the efficient and appropriate application of weathering steels in Australian bridges. It also provides guidance to achieve the expected performance of weathering steel structures in Australia, to realise their planned life span.

This publication covers the design, construction, inspection, maintenance and even rehabilitation of weathering steel, should corrosion rates exceed those anticipated at the design stage, as well as discussing the limitations on the use of weathering steel.

Comments and queries on the guide should be referred to HERA.

Figure 1-1: Kanunnah Bridge over the Arthur River in Tasmania, built in the 1970's in a C2 corrosivity zone

2. Background to Weathering Steel

2.1 What is Weathering Steel?

Weathering steel, or to use its technical title of "structural steel with improved atmospheric corrosion resistance", is a high strength low alloy structural steel that, in suitable environments, may be left uncoated because it forms an adherent protective rust "patina" that minimises further corrosion. The alloys added to weathering steel compose approximately 2% of the steel make-up with specific alloying elements such as copper, chromium, silicon, phosphorus and in some cases nickel. The additional alloying does not diminish the structural capability of the steel, with the steel offering strength, ductility, toughness and weldability suitable for bridge construction, and is covered by the Australian Standard AS/NZS 3678.

All structural steel corrodes, at a rate that is governed by the access of moisture and oxygen to the metallic iron. As this process continues, the surface oxide (rust) layer becomes a barrier restricting further ingress of corrodents to the base metal, and the rate of corrosion slows down.

The rust layer formed on most conventional carbon-manganese structural steels detaches from the metal surface after a period of time and the corrosion cycle commences again.

Hence, the corrosion rate progresses as a series of incremental curves approximating to a straight line, the slope of which depends on the aggressiveness of the environment.

The weathering steel rust patina is initiated in the same way but in a non-marine environment, due to the alloying elements in the steel, it produces a stable corrosion nano-layer (Kimura 2005) that adheres to the base metal and is much less porous.

Figure 2-1: REDCOR® weathering steell Patina on Pestells Lane Bridge, Princes Hwy, NSW

This layer develops under conditions of alternate wetting and drying to produce a protective barrier which impedes further access of oxygen and moisture. Eventually, if this barrier is sufficiently impervious and tightly adhering, the corrosion rate will be greatly reduced. The resulting reduction in corrosion rates is illustrated in Figure 2-2.

In a suitable environment this stable condition may be reached within 8 years, or more, depending on the local environmental factors. At this stage the metal is then protected from significant future corrosion by the rust patina. Assuming that there is no significant change in the environment, and with regular inspection to determine and treat any isolated problem areas if they occur, the potential life of a weathering steel bridge is expected to be more than 100 years.

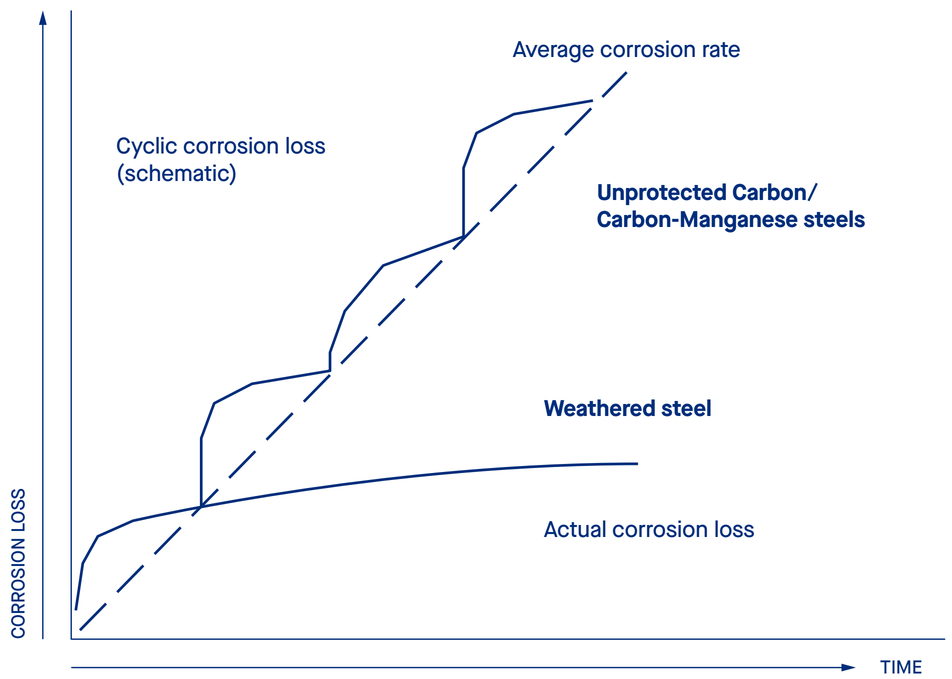

Figure 2-2: Schematic comparison between the corrosion loss of weathering and carbon steels

2.2 Benefits of Weathering Steel

2.2.1 Cost Benefit

Depending on the market price of weathering steel, it may initially be comparable or more expensive than carbon steels, up to 2 to 6% (Kogler, 2015); however, cost savings from the elimination of the protective coating system typically outweigh the additional material costs. The total life cycle cost of a weathering steel bridge could be up to 30% lower than a conventional coated steel alternative (El Sarraf & Mandeno 2010) and (AISI, 2020).

In most cases, a weathering steel solution is cost-competitive with an optimised coated steel alternative for most inland environments.

2.2.2 Reduced Construction Time

The total construction period is reduced as both shop and site painting operations are eliminated or minimised, to the advantage of the contractor and ultimately the client.

2.2.3 Reduced Whole-of-Life Cost and Duration of Maintenance Work

The main advantage of using weathering steel in bridges is the significant reduction of maintenance over the life of the bridge. When correctly detailed, periodic inspection and cleaning should be the only maintenance required to ensure the bridge continues to perform satisfactorily. Since a protective coating is not normally required, the cost of inspection, cleaning and, in some cases, the occasional remedial treatment of limited areas is usually considerably lower than the costs of regular maintenance and recoating of a fully coated structure. This greatly reduces the impact on the environment and indirect costs, such as those resulting from traffic management and traffic delay, caused by providing site access while coating maintenance activities are carried out.

2.2.4 Environmental Benefits

The application and maintenance of protective coatings requires suitable environmental considerations, such as containment of abrasive blast cleaning residue and of industrial paint solvent and overspray. Hence, the omission of a protective coating by using weathering steel, yields both environmental and sustainability benefits.

2.2.5 Attractive Appearance

The protective mechanism of weathering steel in bridges is the formation of a stable film of a rust patina. Once fully formed and weathered, the appearance of this film is uniform, usually of a dark brown or purple colour. This colour can blend nicely with the environment and improves with age.

2.2.6 Health and Safety Benefits

Health and safety issues relating to the application or removal of initial and subsequent coatings are avoided, and safety issues associated with inspection and maintenance access are reduced, thus implementing Safety in Design principles. For example, the need to access the confined space inside closed box girders is minimised.

2.3 Where to use Weathering Steel?

As with other forms of construction materials, there are certain environments that can lead to durability problems. The performance of weathering steel in some environments may also not be satisfactory and its use in these environments should be avoided.

In general weathering steel can be used in C1 to C3 corrosivity categories where the first-year rate of corrosion is less than 50 µm/year, as discussed below. When the location for weathering steel is being considered near a marine environment, section 2.3.1 Marine environments should be applied.

2.3.1 Marine Environment

Exposure to high concentrations of chloride ions will negatively affect the patina formation. These ions can be deposited from airborne marine salts in aerosol originating from breaking waves at sea or on the shoreline, or from salt fogs, and will affect the patina formation. The hygroscopic nature of these salts can prevent sheltered surfaces in such environments from fully drying and thus stop the formation of the rust patina, thereby resulting in the weathering steel continuing to corrode at a rate similar to mild steel.

Evidence of a higher corrosion rate and a delayed, or even no, formation of the protective patina has been documented for unwashed and sheltered surfaces (i.e. microclimatic effects), as well as in crevices, on weathering steel structures in coastal locations (Morcillo 2013). As seen in Figure 2.3, of weathering steel panels within 5m from each other, on the right the panels are rain washed with well adhering patina forming, while on the left, the panels are unwashed and sheltered with the patina struggling to form.

Figure 2-3: Examples of weathering steel in a coastal environment

Thus, near breaking surf, continuous deposition of salt means that conventional weathering steel should not be used in such aggressive environments. In less aggressive marine environments, there is less chloride deposition and on boldly exposed surfaces, rain will normally wash chlorides from the surface and the protective patina will form.

Therefore, when determining the suitability of using weathering steel in a given location, the atmospheric corrosivity assessment needs to consider both the macroclimate, in accordance with AS 4312; as well as the microclimate effects, as described in the Clause 2.3 of AS 4312.

Details regarding humidity levels, wind strengths and wind directions, which assist in determining the macroclimate and microclimate for a specific location, can be obtained from the Australian Bureau of Meteorology site under climate data.

Based on the findings of the Morcillo review of weathering steel performance data, it is recommended that weathering steel should only be used in areas where the maximum first year corrosion rate (taking into account both the macroclimate and microclimate effects on sheltered surfaces as discussed by Francis, 2020) of mild steel is less than 50 µm/yr. This is equivalent to a rain-washed surface in atmospheric corrosivity category C3 (Medium) to ISO 9223 and AS 4312; as shown in Table 2-1. Therefore, if the determined surface specific atmospheric corrosivity is C4 (High) or above, weathering steel should not be used.

Table 2-1: Atmospheric corrosivity categories to ISO 9223, and description of typical environments as per AS 4312

Corrosivity Categories	Description	First Year Corrosion Rate for Steel (µm/year)	Typical Environment
C1	Very Low	<1.3	Dry indoors
C2	Low	1.3 to 25	Arid/rural/urban inland
C3	Medium	25 to 50	Coastal or light industrial
C4	High	50 to 80	Sea-shore (calm)
C5	Very High	80 to 200	Sea-shore (surf)
CX	Extreme	200 to 700	Shoreline (severe surf)

Generally, weathering steel can be used in locations that are more than 500 metres from calm salt or brackish water such as in sheltered bays or 2 km from the open seacoast, where the maximum first year corrosion rate (taking into account both the macroclimate and microclimate effects on sheltered surfaces) of mild steel is less than 50 µm/yr. Examples of where weathering steel may be used, is given in Table 2-2.

It should be noted that as shown in Figures A1 to A6 of AS 4312, the atmospheric corrosion maps of Australia, that the C4(High) corrosivity environment is typically around 1km from the shoreline/open seacoast. In some areas, namely sheltered bays, it doesn't exist. As such, as an inbuilt factor of safety to account for locations where the C4(High) environment extends beyond 1km, a 1km "exclusion" zone is added from C4(High) boundary. Thus, in most locations the above 2 km limit from the shoreline is met, while in others, that extends a bit further.

Table 2-2: Examples of where weathering steel may be used, in relation to distance from the shoreline, modified from Table 4-1 of AS 4312

Location	Example	Minimum Distance from Shoreline*
Temperate Surf	Sydney, Wollongong, Newcastle, Gold Coast	2KM
Temperate semi-sheltered	Adelaide, Brisbane, Perth	
Temperate quiet	Melbourne, Hobart	500M
Tropical quiet	Cairns, Townsville, Mackay	

* Note: Any location within 1 kilometre of the C4(High) corrosivity boundary, weathering steel should not be used.

This was confirmed by a 15-year investigation by BHP on the relative performance of their weathering steel plate (AusTen Type A) compared to conventional steel on outdoor exposure racks located in different parts of Australia (Badger & Wallace 1988), as well as (Li and Marton, 2021) and (Morcillo 2013). It was determined that in the certain locations, that sheltered surfaces (i.e. the microclimate) can be taken as being in C3 (Medium) corrosivity category, even though the site macroclimate was also identified as being C3 (Medium). The findings of this review are discussed in detail in (Mandeno, Francis and El Sarraf, 2024).

In the case that weathering steel is being considered for sites where the microclimate corrosivity is borderline high C3 (Medium)/ low C4 (High), and less than the distances given herein; then determination of the actual site-specific corrosivity environment (including those of unwashed and sheltered surfaces), with a minimum of 1 year record, is required. This quantitative assessment should be undertaken by an experienced corrosion engineer or scientist, having the minimum qualifications given in Section 3.12. It is recommended that the 'coupon weight-loss' technique, to ASTM G1, be used to determine the site- specific corrosivity, rather than testing for salt and sulphur dioxide levels in the atmosphere. Guidance is available for conducting the 'coupon weight – loss' test in Section 5.5.

Alternatively, the 3 monthly qualitative Wire-on-Bolt method, to ASTM G116, could be considered.

Site testing of corrosion rates for sites where AS4312 does not provide guidance for the corrosion category of the location and would be considered different to areas displayed on the existing maps, then testing should be conducted as per the guidance given above.

Therefore, when assessing the site atmospheric corrosivity category in Australia (as per Figures A1 to A6, Clause 3.2 and Table 4-1 of AS 4312) to determine the suitability for the use of weathering steel, the following assessment guidelines should be used:

- Any location in a C4, C5 or CX category or within 500 metres of the shoreline, whether the seas are temperate semi-sheltered, quiet (calm) or tropical quiet (calm), weathering steel should not be used
- Any location within 1 km of the C4(High) corrosivity boundary, weathering steel should not be used
- Weathering steel should be acceptable for all other locations (C2, C3) in Australia that are further inland than these limits
- However, if actual measurements or local features show high corrosivity, then these override these guidelines. These guidelines apply to both boldly exposed and sheltered micro-climates

2.3.2 Localised Adverse Conditions

Contaminated sprays from roads under wide bridges create "tunnel-like" conditions that should be accounted for when the clear height is less than 5.3m.

"Tunnel-like" conditions are produced by a combination of a narrow underpass road with minimum shoulders between vertical retaining walls, and a wide bridge with minimum headroom and full height abutments. Such situations may be encountered at urban/suburban grade separations. The extreme geometry prevents roadway spray from being dissipated by air currents, and it can lead to increased contaminated spray on the bridge girders.

Leaking expansion joints can be caused by faulty seals exposing bridge members to contaminated runoff water; resulting in a higher time of wetness and increased risk of continued corrosion of the weathering steel surface.

Section 3.8 covers detailing solutions to avoid localised durability issues.

De-icing salts used on roads both over and under weathering steel bridges may also lead to problems. These includes leaking expansion joints where salt laden run-off flows directly over the steel and salt spray from roads where "tunnel-like" conditions are created. Both of these cases are much less of an issue in Australia than they are in some other countries, where sodium chloride based de-icing salts are spread on roads, to make driving safer in winter months.

2.3.3 Continuously Wet / Damp Conditions

Alternate wet/dry cycles are required for an adherent patina to form. Where these cannot occur, due to excessively damp conditions, a corrosion rate similar to that of conventional carbon-manganese steel may be expected. Such conditions are to be avoided.

This means that, in addition to the marine environment guidance given in Section 2.3.1, weathering steel members must not be immersed in water, be in contact with soil or be covered by vegetation. There must also be a minimum headroom of 2.5m for crossing over water not subject to significant wave action.

Overseas practice is to limit weathering steel use to regions where the 'time of wetness' is < 60% of the total time. ('Time of wetness' for weathering steel applicability is defined by ISO 9223 as when the relative humidity (RH) exceeds 80% at the site). This is based on the classification of time of wetness greater than 60% as being "very damp for corrosion purposes" by ISO 9223 (Category T5 to ISO 9223). Hence, the RH level at the site should be less than 80% for more than 40% of the year.

Where the average time of wetness is greater than 60% and up to 70%, as shown in the Australian Time of Wetness map given in Appendix B, the following additional restrictions apply:

- the bridge must not be shaded from sunlight between the hours of 9am to 3pm by permanent obstructions such as surrounding hills, at any time of the year.
- the bridge must be located so there is unobstructed airflow past the steelwork.
- the surrounds must be constructed so there is no vegetation taller than grass within 3 metres of the bridge steelwork.

Time of wetness' for weathering steel applicability is defined by ISO 9223 as when the relative humidity (RH) exceeds 80% at the site at temperatures greater than 0°C. Overseas practice is to limit weathering steel use to regions where the 'time of wetness' is < 60% of the total time. This is based on the classification of time of wetness greater than 60% as being "very damp for corrosion purposes" by ISO 9223 (Category T5 to ISO 9223).

2.3.4 Atmospheric Pollution

Weathering steel should not be used in atmospheres where high concentrations of corrosive chemicals or industrial fumes, specifically sulphur dioxide (SO_2), are present. Such environments with a pollution classification above P3 (SO^2 > 250µg/m^3 concentration or 200mg/m^2/day deposition rate) to ISO 9223, should rule out the use of weathering steels.

In Australia, this level of SO_2 may occur within the vicinity of large industrial sites such as at Mt Isa in Queensland and Port Pirie in South Australia.

The other risk to consider is the accumulation of coal dust on the weathering steel surfaces, e.g. from the rail coal hoppers passing over or under the bridge. Coal dust contains pyrite, which is an iron sulphide that is released from the coal seam during mining (Chou, 2012). This sulphur content of coal is corrosive, as sulphur dioxide, produced by the oxidation of pyrite, reacts with hygroscopic constituents (such as chloride ions found in marine aerosol) and in turn moisture, generating sulfuric acid (H_2SO_4), which attacks the metal.

However, recent research (Crespo 2020) indicates that a 10% solution of H_2SO_4 actually accelerates the patina formation and is used for treating and repassivating weathering steel sculptures. Hence it appears that a low concentration of sulfuric acid may be beneficial in the absence of chloride contaminants.

Therefore, for weathering steel rail bridges on coal routes, it is prudent to specify the use of weathering steel in only C1 (Very Low) and C2 (Low) corrosivity environments, as shown in AS 4312; until further evidence is found to justify its use in C3 locations.

This is because in C2 (Low) corrosivity environments, not only is the atmospheric corrosion rate relatively low, but those areas also have low marine influence. This in turn reduces the risk of the contamination by hygroscopic constituents, which reduces the risk of sulfuric acid being generated.

Figure 2-4: Sydney Gateway Bridge featuring REDCOR® weathering steel

3. Design and Detailing

3.1 Design Codes

In Australia, the design for weathering steel bridges should be undertaken in accordance with the relevant parts of Australian Standard AS 5100, especially Part 6.

However, there are a number of requirements, mostly related to detailing and suitability of environment, for weathering steel. These are outlined below.

3.2 Material Specification

BlueScope supplies REDCOR® weathering steel to the Australian Standard AS/NZS 3678, with the typical material properties given in Table 3-1.

Table 3-1: Properties of AS/NZS 3678 weathering steel

Steel Grade[1]	Thickness of Material(t) (mm)	Minimum Yield Stress (MPa)	Minimum Tensile Strength (MPa)	Charpy Impact Toughness[2]
WR350B	10 to 80	340	450	–
WR350LOB				27 Joules @ 0°C
WR350L20B				27 Joules @ -20°C
WR350B MOD 400	10 to 12	400	480	–
WR3500LOB MOD 400	12 to 20	380		27 Joules @ 0°C
WR400L20B	20 to 70	360		27 Joules @ -20°C

Notes to Table 3-1

1. The Australian steel grade designations mean:
   WR: represents "weather-resistant" (i.e. improved atmospheric corrosion resistance or weathering steel).
   – 350 & 400: represents the Nominal Yield Strength in MPa.
   – L0: relates to the impact test temperature (0°C).
   – L20: relates to the impact test temperature (-20°C).
   More detail is found in the Australian Standard AS/NZ 3678.
2. Using Charpy 2mm V-notch impact test specimens.
3. Thicknesses down to 3mm are available as coil plate to AS/NZS 1594 HW350A.

3.3 Allowance for Loss of Thickness

Assuming that the protective rust patina is formed, even with the reduced corrosion rate, an allowance for the expected section loss over the design life of the bridge should be considered. The corrosion allowance is added to each exposed surface and this added thickness for the corrosion allowance cannot be used in the calculation of the structural capacity of the member.

Table 3-2 outlines the corresponding corrosion allowance for a bridge design life of 100 years, as specified in AS/NZS 5100.6.

Table 3-2: Corrosion allowance (use with Table 2-1)

Atmospheric Corrosion Classification (ISO 9223)	Weathering Steel Environmental Classification	Corrosion Allowance (mm/exposed face)
C1, C2	Mild	1.0
C2 (within 1 km from the sea)	Medium	1.5
C3	Medium	1.5
C1, internal	Interior (Box girders)	0.5

Notes to Table 3-2

- No allowance is required for interior faces of hermetically sealed hollow sections.
- Internals of box (tub) girders supporting concrete decks should be classified as C2, unless designed to prevent water ingress.
- Allowances to apply to all fillet and partial penetration butt welds.
- Full penetration butt welds do not normally require any additional corrosion allowance, because an allowance has been applied to the parent material thickness.
- No allowance is normally made for weathering steel bolts. See Section 3.5.4.
- Allowances apply to all other structural elements, including stiffeners and bracing etc.
- Weathering steel should not be used for ISO 9223 Classifications C4 and C5 or for certain Classification C3 sites as per the guidance given in Section 2.3.1.
- If required, additional guidance on the corrosion allowance can be sought by contacting HERA.

However, based on Section 2.1 of ECCS guidance (ECCS, 2021), and specifically its Figure 2-2, it implies that in a C3 environment, weathering steel has the potential to achieve a 300-year design life assuming a 1.5mm corrosion allowance, based on a 0.005mm/annum corrosion rate. Thus, illustrating its potential as a long life, sustainable solution.

3.4 Design

3.4.1 Global Analysis

Since the global analysis for member actions and deflections is usually not particularly sensitive to the exact thickness of the steel sections, the given member sectional area and second moment of area, may be used.

3.4.2 Detailed Design

Although it is unlikely that, at any given section and time during the life of the bridge, the entire exposed perimeter is uniformly corroded to the corrosion allowance given above; the member's net thickness and area, taking into account the corrosion allowance, should be used when designing the members structural capacity.

3.5 Bolted Connections

For bolted connections, the following topics should be considered.

Figure 3-1: Bolted Connection on Sydney Gateway Project SB21, Mascot, NSW

3.5.1 Material Selection

The standard bolts used in bridges, are usually Property Class 8.8 structural bolts to AS/NZS 1252, which are supplied as hot-dip galvanized. While, these can be used in weathering steel bridges, the dissimilar metals in contact with each other (i.e. zinc and weathering steel), will result in the depletion of the zinc and faster corrosion of the bolt over time; as discussed in Section 3.11. Furthermore, they will look distinctly different, especially as the galvanised bolts are coated with a dissimilar colour to the weathering steel. This in turn will introduce a maintenance issue, due to the need to refurbish the coating on the bolts over the design life of the structure.

Another unsuitable option is unprotected "black" bolts, with those manufactured to AS/ NZS 1111, which are of lower strength, being Property Class 4.6-mild steel. However, steel shear studs that are embedded in concrete are acceptable as the alkaline environment prevents galvanic action from occurring.

Instead, weathering grade high strength friction grip (HSFG) bolts, nuts and washers should be used. These are available from the United States, Japan or Britain, and now can be sourced in Australia.

Two strengths are available, as discussed in the Installation of Bolted Connections to AS/NZS 5131 (ASI, 2023); these are ASTM A325, Type 3 or ASTM A490, Type 3 bolts; which are consolidated into ASTM F3125. In Australian terminology, the ASTM A325 bolt is equivalent to Property Class 8.8 bolt and the ASTM A490 has a higher strength, with mechanical properties of a Properly Class 10.9 bolt. They must be used in conjunction with the matching higher-grade nuts and washers.

They are also available through Australian distributors from Britain in metric sizes (such as EN 14399-10 and ASTM A325M) and come as M20, M24 and M30, or more commonly available from the United States in imperial sizes shown in Table 3-3 below.

Table 3-3: Conventional weathering steel bolts

ASTM/EN Designation No.	Size Range inclusive, in	Minimum Proof Stress1, kpsi or (MPa)	Minimum Tensile Strength, kpsi or (MPa)	Minimum Yield Stress, kpsi or (MPa)
A325, Type 3	0.5-1.0 (12.7-25.4mm), 1.125-1.625 (28.58-41.3mm)	85 (586 MPa), 76 (524 MPa)	120 (827 MPa), 107 (738 MPa)	92 (634 MPa), 83 (572 MPa)
A490, Type 3	0.5-1.5 (12.7-38.1mm)	120 (827 MPa)	150 (1034 MPa)	130 (896 MPa)
EN14399-10 (PC 10.9)	M24 & M30	(830 MPa)	(1040 MPa)	(940 MPa)

Notes to Table 3-3
The minimum proof stress is the minimum stress in the installed bolt when installed in accordance with ASTM A325 (or Clause H2.5.2 of AS/NZS 5100.6).

3.5.2 Coefficient of Friction

It has been found (Kulak et al, 2001) that for tightly adherent mill scale on the surface of weathering steel at a bolted connection, the connection slips into bearing at a lower shear stress than that of carbon steel with mill scale. The coefficient of friction was $\mu s = 0.2$ for weathering steel versus 0.35 for carbon steel.

However, for high strength friction grip connections, the full removal of the mill scale is required, to achieve a higher coefficient of friction of $\mu s = 0.5$. Therefore, to use this higher friction factor, all faying surfaces should be abrasive blast cleaned using non-metallic grit to Commercial Blast Cleaning as defined under the visual cleanliness standard SSPC SP-6/NACE No 3. See Section 4.4 for further guidance on surface preparation.

Furthermore, the development of an adherent rust film, such as that produced by wetting and drying for several months by exposure to rain or washing with potable water, does not degrade the coefficient of friction. However loose rust or any residual mill scale would impair the performance of the joint, and its removal from faying surfaces by wire brushing or scraping prior to assembly of the connection must be specified.

3.5.3 Crevice Corrosion and Bolts Spacing

Crevice corrosion is an issue with all types of bolted connections, but provided the surfaces are held together in sufficiently close contact it has been found (ECCS, 2021), that problems such as rust staining and pack rust do not arise.

However, it must be recognised that any flexing in service of the connected steel members can open up the joint and lead to the ingress of moisture and dissolved contaminants as a result of capillary action. Hence more stringent requirements on bolt centres and edge distances are required in comparison to joints in a conventional steel bridge.

- Bolt spacing in lines adjacent to plate/section edges should not exceed fourteen (14) times the thickness of the thinnest component, and in any event should not exceed 175mm.
- The distance from the centre of any bolt to the nearest free edge of a plate should not exceed eight (8) times the thinnest component and in any event should not exceed 125mm.
- These requirements given in (SCI 2015) are slightly more stringent than the requirements give in Clause 12.5.2.3 and 12.5.2.4 of AS/NZS 5100.6. If these limitations cannot for any reason be met, either the joint must be protected by a suitable coating or a suitable sealant should be applied around the edge of the joint. Also load indicating and spring washers should not be used due to the risk of crevice corrosion.

Designers should be aware of Issue 94 of “Construction and Technology”, April 2002, which covers an example of a bridge mounted sign failure between the mounting plate and the bridge beam in weathering steel bridges. It gives recommendations for bolts and treatment of the contact surface between the plate and beam, for attachments to the beams, such as for these signs. See Section 7.2 for guidance on sealing of crevices.

3.5.4 Corrosion Allowance for Bolts

There are no specific code requirements for any allowance to be made on the size of bolts. It is reasonable to assume that in a properly detailed bolted connection (see below), the bolt shank may be treated as “not exposed” and hence no allowance needs to be made; the size of the bolt heads and nuts are generally sufficient to accommodate any loss that occurs. If there is any cause for concern (for example poor fit-up or flexible cover plates allowing water to be attracted by capillary action under the bolt head or the nut), local sealing and/or coating is likely to be preferred treatment, since continued wetness could cause corrosion far in excess of any nominal allowance. Maximum bolt pitches to avoid capillary action occurring are given above.

3.5.5 Design of Bolted Connections

Metric sized weathering steel bolts are becoming more readily available, especially in relation to the commonly used bolts sizes on bridges, mainly M24, M30 and M36.

When imperial sized bolts are specified, it is recommended that bolted connections are designed for the M24 bolt, but with 1 inch bolt spacing. This will maximise the procurement options available to the contractor, who can then substitute 1 inch bolts for M24 bolts, without affecting the layout or design of the connection.

Alternatively, if it was confirmed from the beginning of the design process that imperial bolts will be used, it will be more economical to design for the larger 1 inch bolts, due to the additional available capacity in comparison to the slightly slimmer M24 bolts.

Bolts in tension friction and bearing-type connections of weathering steel bridges should be tightened using the part-turn method as outlined in Clause H2.5.2 of AS/NZS 5100.6 and spaced as detailed in Section 2.9; with additional guidance relating to the installation of weathering steel bolts, given in Appendix E.

As discussed, great care also has to be taken to avoid crevices which can admit water at the ends of lapping plates, and similar details.

While the designer can specify joints such that this is unlikely, the fabricator still has to ensure good fit-up with flat plates. As a last resort, sealants are available which will perform adequately with weathering steels to prevent water ingress to such joints; as discussed in Section 7.2.

3.5.6 Thread Lengths of Different Structural Bolts Standards

As outlined in Section 13.5 of the Installation of Bolted Connections to AS/NZS 5131 (ASI, 2023), designers should be aware that there is a significant difference between the ASTM F3125 and AS/NZS 1252 bolt thread lengths (see Table 3-4). ASTM F3125 specifies ASME B18.2.6/M for the dimensions of both metric and imperial structural bolts. The thread lengths in these standards are less than 2x diameter for most sizes which requires that connection details be accurate to avoid complications on installation. The benefit of this is that designers can utilise the stronger shear strength of the bolts as the shear plane is more likely to cross the unthreaded shank rather than the threads.

AS/NZS 1252.1 is based on EN 14399 HR type assemblies and as such have similar thread lengths. The thread lengths are longer and increase with increasing bolt length, this gives installers more flexibility when there are differences between the connection detail grip length and the actual grip length.

Table 3-4: Thread Length comparison between ASME and AS Standards

Metric	ASME B18.2.6/M	AS/NZS 1252.1 / EN14399 HR			Imperial	ASME B18.2.6
		l<125	125<l<200	l>200		
mm	mm	mm	mm	mm	Inch	Inch (mm)
M24	41	54	60	73	1	1.75 (45)
M30	49	66	72	85	1-1/4	2.00 (50)
M36	56	78	84	97	1-1/2	2.25 (57)

3.5.7 Preloading Suitability

Structural bolting product standards are intended to be used in conjunction with the design, fabrication and erection standards of the same standards organization. For example, AS/NZS 1252.1 is intended to be used with AS/NZS 5100.6 and AS/NZS 5131. When other product standards are used it is important that designers and installers are aware of the different requirements that should be demonstrated before bolts are installed into steelwork.

Preloading suitability testing demonstrates that the bolt, nut, washer assembly can reliably achieve the specified minimum tension requirements with tensioning methods to be used for installation. AS/NZS 1252.1 and EN 14399 demonstrate preloading suitability through the designation of an assembly lot which is a unique identifier of the bolt, nut, washer assembly. AS/NZS 1252.1 structural assemblies are required to be tested to Appendix D “Assembly Testing”, with the results of this test on the bolt assemblies test certificate.

ASTM F3125/M does not require designation of an assembly lot unless specified by the purchaser. In most cases test reports will be made available for the individual components, however, no suitability for preloading test will be available. The intent of the ASTM method is that bolt assemblies are tested in a bolt tension measurement device such as a Skidmore-Wilhelm device, on site before bolts are installed (ASI, 2023). Results of this testing are observed and retained by the bolting supervisor for the individual job.

Bolt load indicating devices are not a common tool on Australian construction sites particularly in large diameters. Where ASTM F3125/M structural bolts have been specified it is recommended to also specify that ASTM F3125/M Annex A2 “rotational capacity testing”, as outlined in Section 13.7 of (ASI, 2023) be required for the bolt assemblies to be used on site. Since, ASTM F3125 does not require the assembly lot to be sourced from the same supplier, where that is the case, it is recommended that each configuration of bolt nut and washer should be tested. Sampling should be in accordance with ASTM F1470. This method of demonstrating preloading suitability is in line with current Australian practice.

Weathering Steel bolt assemblies to ASTM F3125/M with Annexure A2, as per (ASI, 2023) rotational capacity testing is available from Australian bolt suppliers.

3.5.8 Bolt Storage and Condition

AS/NZS 5131 states “Fastener components shall be protected from dirt and moisture in closed containers at the location where the bolts are to be installed. Only the required number of fastener components to be installed shall be taken from protected storage at any one time.” Weathering steel structural bolts will weather quickly if not stored correctly. Bolts where red rust has started to form are likely to have a significant change to the torque tension relationship and pose a risk of thread stripping or failure before reaching minimum specified tensions.

Weathering steel bolts will be packaged with a coating (mainly phosphate and oil) to protect the bolts during shipment and storage as well as lubricate the bolts during installation. Where this oil has been removed or weathering (red rust) is observed on the bolts it is likely that the torque required to pre-tension these assemblies will result in a torque failure or thread stripping of the bolt. Where bolts appear dry or weathering has begun it is recommended to apply a bolt specific lubricant such as stick wax before attempting to pre-tension.

Bolts which have been removed from their packaging but not installed are difficult to trace and risk incorrect bolt lengths being installed into connections.

Further guidance on the handling of weathering steel bolts is given in Section 13.6 of (ASI, 2023).

3.6 Welded Connections

Welding is of critical importance in the fabrication of steel structures as welds are usually the most critical parts of the structure. Therefore, great attention should be given to the selection of welding consumables to achieve the desired characteristics of resistance to atmospheric corrosion and the colouring pattern of the welds; especially when a client has particular expectations for the appearance of a fabrication. Welding must comply with the quality requirements of the applicable welding standard and product specifications. In Australia and New Zealand welding of weathering steels is required to comply with AS/NZS 1554 Part 1, 2 or 5, and execution requirements as applicable for the Construction Category (CC) of AS/NZS 5131 as specified for the structure or the section.

It is the responsibility of the fabricator to comply with all applicable requirements of the Construction Category (CC) specified for the structure including qualification of Welding Procedure Specifications (WPS)s.

All joints, including fillet welds, should be continuously welded to avoid moisture and corrosion traps such as crevices.

Clause 3.9.1 of AS/NZS 5100.6 includes the recommendation that for rail bridges, and road and pedestrian composite box girder bridges, the total weld design throat thickness (DTT) of web-to-flange welds shall be not less than the thickness of the web.

Where used for I-beam girders, fillet welds shall be provided on both sides of the connecting flange or web plate. Cost savings can be achieved by specifying fillet welds instead of complete penetration butt welds, as the latter can be several times more expensive compared to fillet welds.

Complete penetration butt welds can be substituted by fillet or deep penetration fillet welds. By optimising (e.g. enlarging) the fillet weld size or weld penetration (DTT), these welds can be designed to prevent weld root cracking, achieving fatigue performance equivalent to the butt welds. Fillet welds can also be substituted by partial penetration butt welds with a superimposed fillet weld component with the total weld throat thickness (DTT) equivalent to that of the recommended fillet welds. Deep penetration fillet welds can be considered in conjunction with the use of automated welding processes such as SAW.

Clause 12.6.6.1 of AS/NZS 5100.6 limits the use of incomplete-penetration butt welds to the longitudinal joints to connect the elements of built-up members, such as girders and columns only. Incomplete-penetration butt welds should not be used to transmit tensile or compressive loads, or bending moments about the longitudinal axis of the weld.

I-beam bridge girders welded by fillet and butt welds are categorised in the same fatigue detail in accordance with Table 13.10.1(c) of AS/NZS 5100.6 provided they are executed using the automatic or fully mechanised welding process e.g., SAW. For improved fatigue performance, the web-to-flange fillet welds in built-up sections should have a uniform weld profile with no stop-starts or with stop-starts smoothly blended and inspected.

Site welding should be generally avoided as it requires local grit blasting or grinding after welding which may lead to differences in appearance of the welded joint from the rest of the steelwork. If site welding is to be used, this must be designed and detailed for, from the start of the design stage and preferably not made as a late change in the design and detailing process, once the general details and layout are established.

A prerequisite for obtaining identical mechanical properties in the weld and in the base material is the use of suitable welding consumables and the choice of appropriate welding conditions according to AS/NZS 1554 Part 1 or 5. The Standard however does not specify requirements for every aspect of the welding; there are various matters that need to be resolved between the designer and the fabricator. This includes the possible use of C-Mn unalloyed welding consumables for single run welds and type and amount of welding inspection. Appendix D in AS/NZS 1554.1 provides the list of “Matters for resolution”; a normative part of the Standard that both designers and fabricators must consider before the start of the job.

BlueScope also have an advisory Technical Note on the Welding of Weathering steel (Supplement to Technical Bulletin TB 26) that provides guidance on the welding of weathering steels.

See also Section 4.3 and note to Table 3-2.

3.7 Fatigue

Concern has been expressed that weathering steel bridges may exhibit lower fatigue performance than those in ordinary structural steel. This view comes from the fact that corrosion forms pits from which fatigue cracks might initiate easily, with the corrosion then following the crack and hence increasing the speed of propagation. Many tests have been carried out worldwide to investigate this, and whilst the results are not all in full agreement, there seems to be a general consensus that:

a) After the weathering process has occurred, parent weathering steel (e.g. constructional details 1-3 from Table 13.5.1(A) of AS 5100.6), will have lower fatigue strength than parent non-corroded steel, because of the greater surface roughness under the corrosion layer.

b) Fatigue failures in bridges are almost always initiated at a point of geometrical discontinuity or stress concentration such as a weld: this gives a much greater reduction below the parent steel fatigue strength than does the presence of corrosion pits in weathering steel. It appears that, provided the detail category to Table 13.5.1(A) of AS 5100.6 of the critical detail is 100 or lower, the use of weathering steel will not cause any reduction of fatigue life. Even if the detail is detail category 112, degradation would be minimal.

c) Tests which show worse behaviour of joints with low fatigue detail category in weathering steel have apparently always been carried out in very adverse testing environments (for example continuously sprayed with salt water). Weathering steel should not be used for bridges in such environments. Therefore, provided that the guidelines on suitable environments given in Section 2.3 are observed, the results of such tests will not be applicable.

Even though in practice welded weathering steel bridges will not have a lesser fatigue performance than those of coated steelwork, it is necessary to design the details to be as fatigue-resistant as practicable. As is discussed in Section 5.6, fatigue cracks in weathering steel bridges may be harder to detect than those in coated carbon steel bridges.

3.8 General Structural Detailing

The first principle to be stated when detailing a weathering steel bridge is that good structural detailing practice should be used. In this regard, in the correct environment, a bridge whose details would give no problems in coated carbon steel bridge would behave entirely satisfactorily if designed in weathering steel.

However, the following details are particularly important to be aware of:

3.8.1 Drainage

Surfaces of a weathering steel bridge should not be permanently wet or damp. Hence, even if the general environment is satisfactory, it is important to ensure by good detailing that a high time of wetness does not occur at any point on the bridge steelwork.

There are a number of ways in which this can be achieved, some of which are illustrated below. Some of these details may be expensive to fabricate or, if in fatigue sensitive areas, may lead to a reduced fatigue life. A designer must carefully weigh the relative advantages and disadvantages, taking all factors into account, before selecting a detail.

Weathering steel bridges should be detailed to ensure that all parts of the steelwork can dry out, by avoiding moisture and debris retention and by ensuring adequate ventilation and any ponding of water.

Common practices in this regard, taken from (ECCS, 2021) are to:

- Grind flush weld details which may cause water traps (Figure 3-2)
- Provide 50mm copes where stiffeners are attached to the bottom flange
- Avoid closely spaced girders to aid ventilation (Figure 3-3). This is also a more economical layout
- Avoid overlaps, pockets, faying surfaces and crevices, which can collect and retain moisture (Figure 3-4 to 3-7)
- Hermetically seal hollow sections, or provide adequate access, drainage, and ventilation
- Provide slopes or cross fall to ensure water runoff. Provide drip plates to direct runoff away from bearings or where staining could occur
- Where possible web plates of box girders should extend 20mm below the bottom flange, including welds to form a drip detail (Figure 3-8)

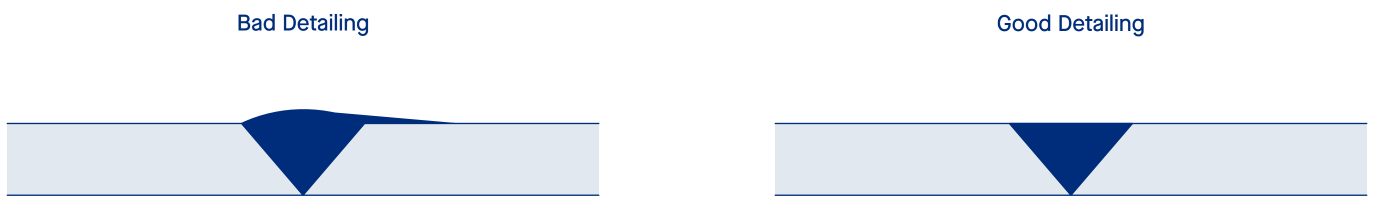

Figure 3-2: Grinding flush of welds which otherwise form water traps

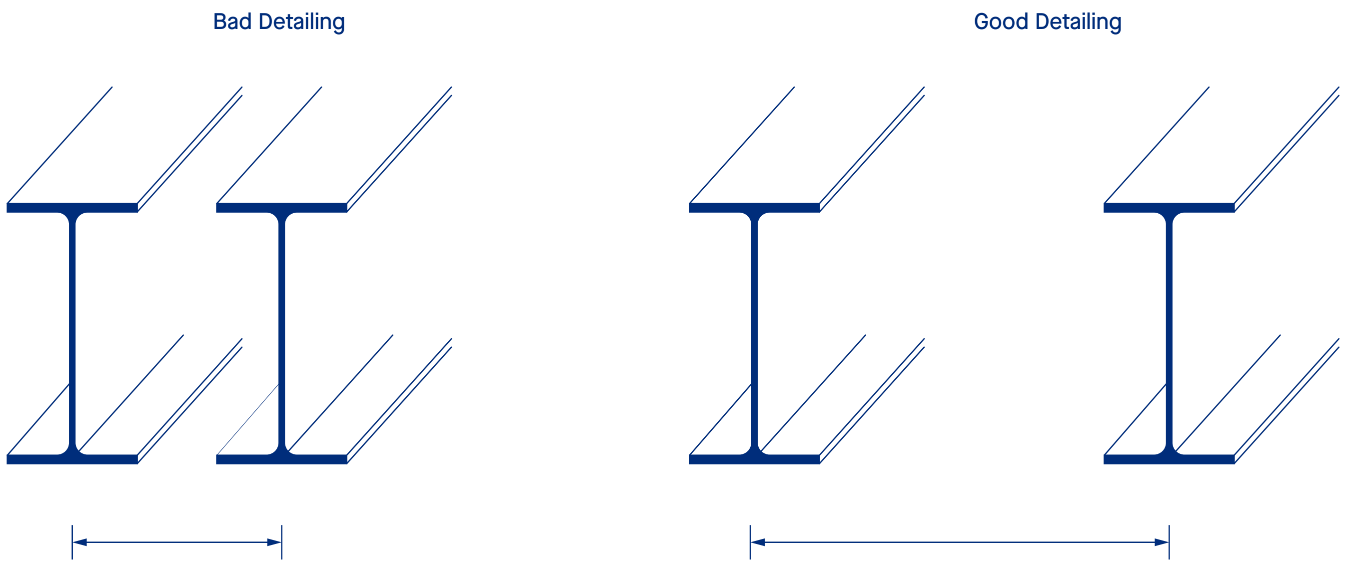

Figure 3-3: Spacing of Girders

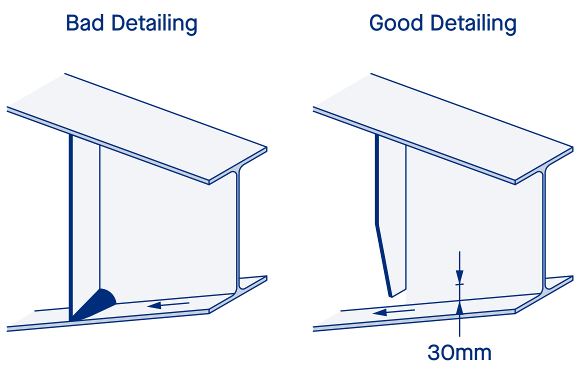

Figure 3-4: Curtailing transverse web stiffeners to allow drainage below

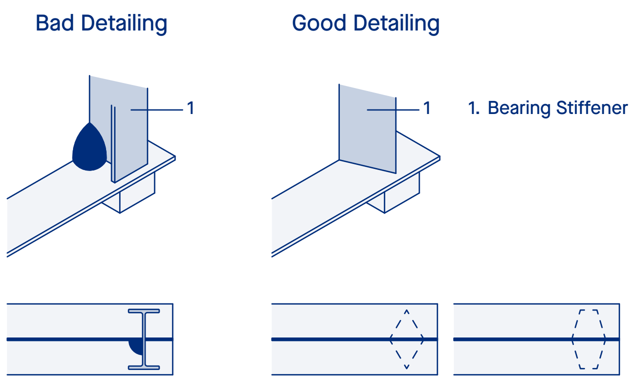

Figure 3-5: Optimum choice of transverse stiffener shape

Drip details must be provided at the caps or edges of concrete decks in composite weathering steel bridges (Figure 3-6). If possible, the overhang of the bridge deck over the steel girder should be at least equal to the height of the girder to avoid direct wetting by rain. This also applies to the bridge caps of concrete decks in composite weathering steel bridges.

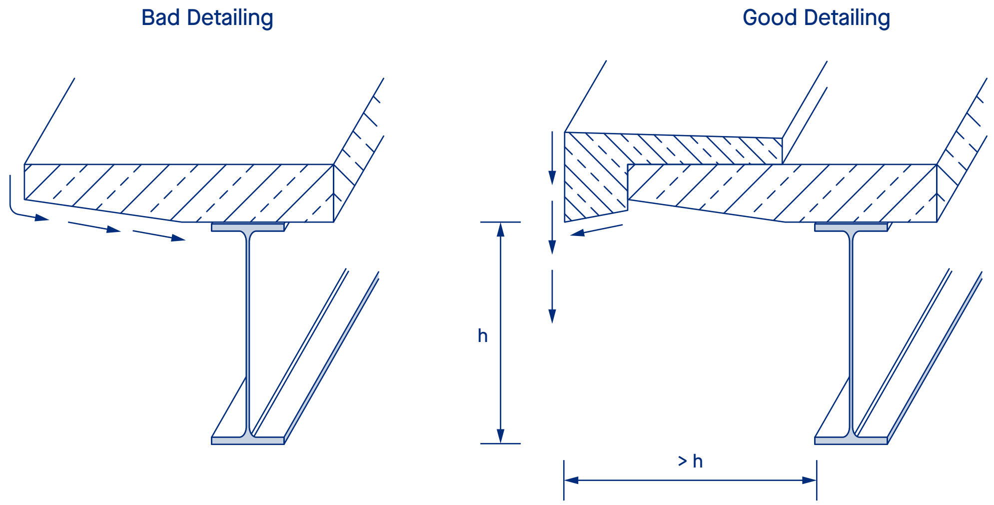

Figure 3-6: Drip details at the caps or edges of concrete decks and overhang

The distance between the end of the girder and the retaining wall of the abutment should be at least 50 cm to ensure sufficient ventilation and drying (see Figure 3-7).

The end of the bridge deck should have a drip edge above the abutment, as the expansion joint above may leak. In addition, a drainage or rain gutter with an appropriate gradient should be planned under the expansion joint or at least a sloped abutment platform with a drainage gutter and pipes, as shown in Figure 3-14.

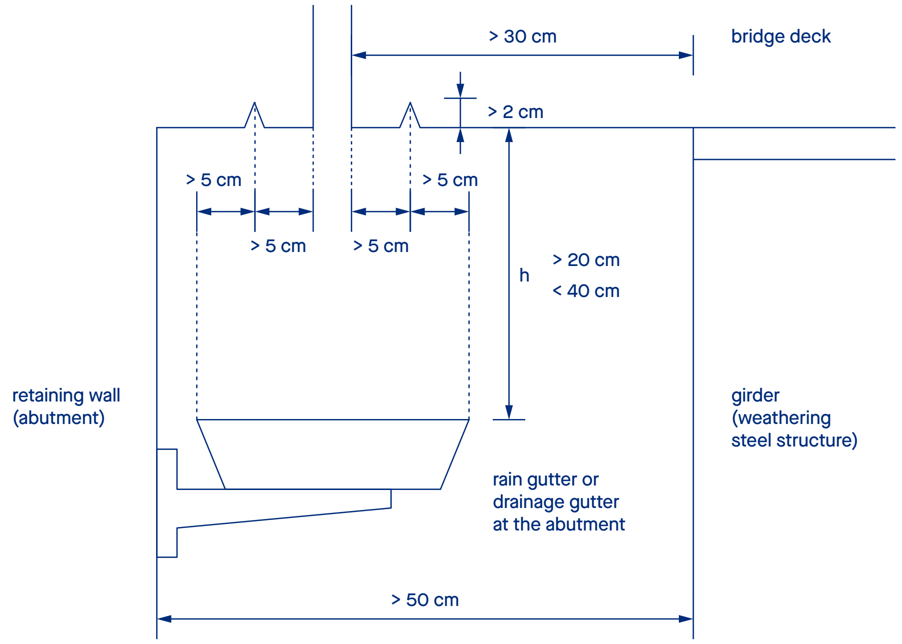

Figure 3-7: Distance between the end of the girder and the abutment

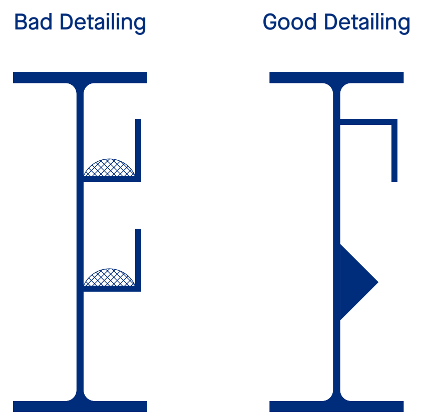

Figure 3-8: Correct orientation of longitudinal stiffeners

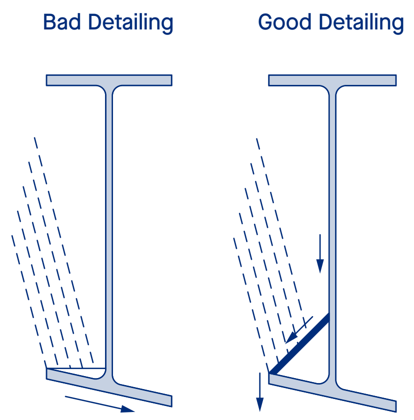

Figure 3-9: Provision of run-off slopes on external flanges

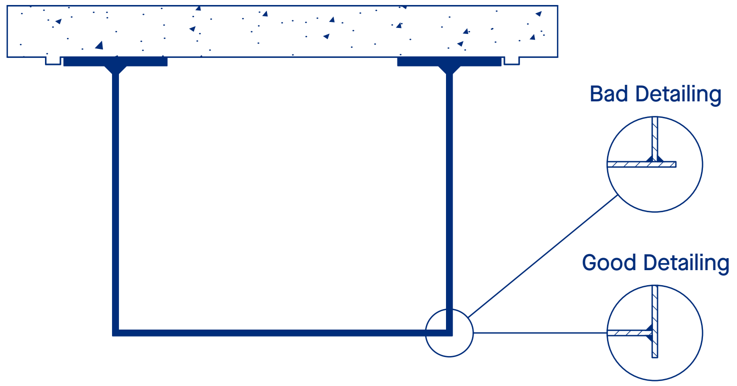

Figure 3-10: Use of box sections

Note: Use box section girders where technically and economically feasible, and ensure that lower flanges do not project horizontally (see Figure 3-8). However internal condensation may occur and adequate internal ventilation and drainage must be provided, especially when supporting an unsealed concrete deck.

3.8.2 Crevices

Crevices should be minimised and where possible, eliminated. Crevices can attract moisture by capillary action, and can be a particular problem for weathering steel bridges, as they are not coated or sealed. Crevices can occur at any point where two surfaces are in contact, and are particularly an issue for bolted connections where plates lap (see further comment on bolted connections in Section 3.5.3, including maximum bolt spacing). If a crevice is not adequately sealed, not only is water attracted into the crevice without much chance of escape, but the corrosion products formed as a consequence have a higher volume than the original material that may result in the distorting or bursting of the connection. Furthermore, the corrosion products themselves will tend to attract further water and thus aggravate the situation.

In cross bracing between girders, use angles "flange upwards" (Figure 3-6; good detailing), and select "K" bracing rather than "X" bracing to avoid crevices at the intersections. If "X" bracing must be used, fill out the intersections with tightly fitting filler plates, as shown in Fig 3-11.

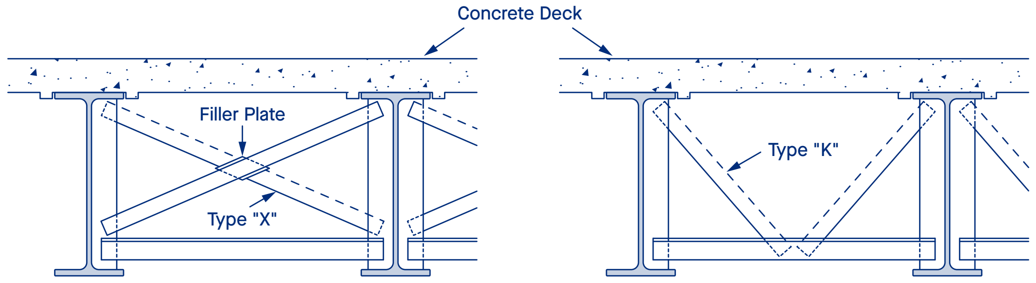

Figure 3-11: Type X and type K Cross Bracing Details

3.8.3 Expansion Joints

Minimise the number of deck joints, as leaky deck joints are one of the most common causes of problems in weathering steel bridges, allowing contaminated water to drop onto the steel below the deck, as well as increase the time of wetness that will affect the patina formation. Therefore, weathering steel should be used for bridges in conjunction with the deck being continuous over intermediate piers (Figure 3-12) and, where possible, where the deck is integral with the abutments, see (El Sarraf et al 2013) for additional guidance.

If deck joints are unavoidable at abutments, give special attention to them to ensure that they do not leak or, if there is any risk of leakage, that they are provided with a positive drainage system whose outlet pipes are of sufficient length to ensure that the discharge water does not spray on to the adjacent steelwork or substructure in any wind condition. The use of drainage items of a non-metallic type is preferable.

Figure 3-13 shows on the left an example of a damaged oxide layer (formed incorrectly). This damage is caused by a humid environment by dirt and water as a result of a leaking expansion joint combined with a badly or not planned drainage system at the abutment. On the right, an example of a local rehabilitation of a damaged area at another bridge (Mamer viaduct in Luxembourg) with a subsequent corrosion protection paint is given.

Figure 3-12: Long span "jointless" bridge

Figure 3-13: Damaged oxide layer due to leaking expansion joint and poor drainage system at the abutment (left) and a local rehabilitation of a damaged area with a subsequent corrosion protection coating

3.8.4 Run-off

Run-off of water from the super-structure and drains should not be permitted to run down the visible external surfaces of the substructure. Until the patina is fully formed, run-off water is liable to contain rust from the weathering process, and unless it is kept away from such surfaces will cause unsightly staining. The drainage of the deck, piers, and abutments requires careful design and detailing to ensure that staining is avoided.

This usually means channelling any run-off water on the tops of piers or abutments, and around bearings to a drain, or drains, feeding into down-pipes which discharge away from the pier or abutment. Particular care should be taken to ensure (by diversion strips or otherwise) that run-off from bottom flanges occurs away from piers. Refer to Figure 3-14 and Figure 3-15 for appropriate drainage in the abutment, and the correct orientation of the drip plate. Also see Figure 3-16 and Figure 3-17 which show examples of good and
bad detailing.

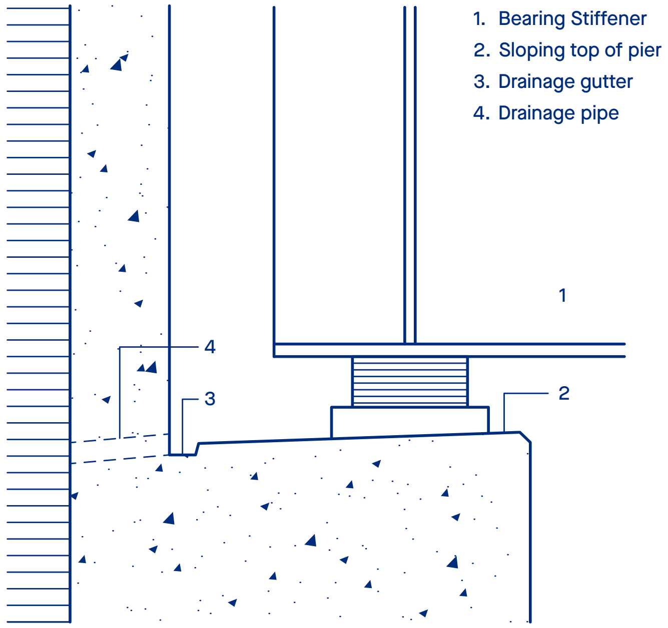

Figure 3-14: Sloped abutment platform and drain

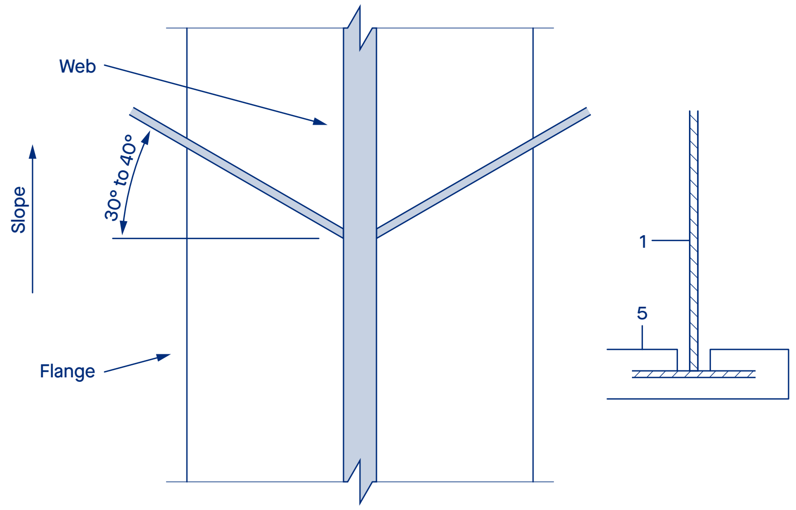

Figure 3-15: Drip plate attached to bottom flange, sloped to prevent debris accumulation. Sloped abutment platform and drain

Figure 3-16: Good detailing producing an abutment free of staining

Figure 3-17: Bad detailing (severe staining)

Concerns were raised about the potential environmental impact of the run-off, due to the release of corrosion product (i.e. rust) into the environment. A study by (Raffo, et al) assessed the concentration of dissolved alloying metal in the run-off over a period of 3 years, which are given in Table 3-5 below. These are given for iron, manganese, chromium, nickel and copper (Fe, Mn, Cr, Ni, Cu) at a marine urban site; given as C3 (Medium), in units of μg/m³.

Table 3-5: Dissolved weathering steel alloying metal concentrations in run-off over a 3 year period

Year	Dissolved Metal (μg/m³)				
	Chromium	Copper	Iron	Manganese	Nickel
1	<0.2	-	447	28	6.1
2	<0.2	-	27	6	2
3	<0.2	-	42	5.5	1.7

When assessing water quality with respect to freshwater and marine ecosystems in Australasia, the Australian and New Zealand Guidelines for fresh and marine water quality (ANZG) are referred to. These guidelines provide authoritative guidance on the management of water quality for natural and semi-natural water resources in Australia.

The guidelines promote a strong emphasis on building conceptual site models of water systems along with providing toxicant default value guidelines for a range of freshwater and marine water ecosystems for both sediments and water. It must be noted that Water Quality Guidelines are not mandatory and have no formal legal status, but that, where appropriate, state, territory or local jurisdictions may incorporate the processes and tools, including the DGVs (Default Guideline Values), provided within the Water Quality Guidelines, into their water quality protection policy and regulatory tools.

Default guideline values (DGVs) are used as a starting point to assess water quality and are relevant for generic applications where site specific data is not available. The DGVs are derived from ecotoxicity testing using a species sensitivity distribution of chronic toxicity data which will protect 80%, 90%, 95% or 99% of species. These guidelines, for both freshwater and marine environments, are referenced within the summary in Table 3-6.

Table 3-6: ANZG ecological receptor dissolved metal acceptable limits

ANZG Ecological Receptor	Species Protection %	Dissolved Metal ($\mu g/m^3$)				
		Chromium	Copper	Iron	Manganese	Nickel
Freshwater	90%	6	1.8	ID	2500	13
	95%	1	1.4		1900	11
	99%	0.01	1		1200	8
Marine Water	90%	20	3		80*	200
	95%	4.4	1.3		80*	70
	99%	0.14	0.3		80*	7

Note on Table:
*Level of species protection unknown – indicative interim working level
ID Insufficient data to derive a relatable trigger value.

By comparing the measured 3-years dissolved metal alloying through run-off with the allowable limits given in the ANZG, it is clear that the environmental impact of “rust run-off” can be taken as being negligible.

Particular care should also be taken to ensure that run-off from bottom flanges occurs away from piers. For this, drip plates can be used in the correct orientation, considering the slope of the bottom flange to avoid debris accumulation, as shown in Figure 3-18. Instead of drip plates, welded seams or other suitable devices, such as glued PVC angles, can also be used as diversion strips.

Figure 3-18: Bad detailing of drip plate. Drip plate improperly set at 90°, creating corner for debris accumulation

The use of drip plates or welded seams can reduce the fatigue strength of the bottom flange due to a lower detail category.

If there is any risk of water accumulation due to the geometry of the drip plates or welded seams, these should be removed or levelled.

To protect piers from run-off, drainage gutters in combination with downpipes (Figure 3-19) can be used instead of drip plates. It should be ensured that the downpipes are dimensioned and protected in such a way that they cannot freeze or become clogged. As an alternative, drip pans (Figure 3-20) made of REDCOR® weathering steel or stainless steel can be planned as given or otherwise retrofitted by welding them to the bottom of the existing girder (El Sarraf and Mandeno, 2020), also see Section 3.11. The drip pan must be designed so that the air can circulate around the weathering steel superstructure and bearing can be exchanged.

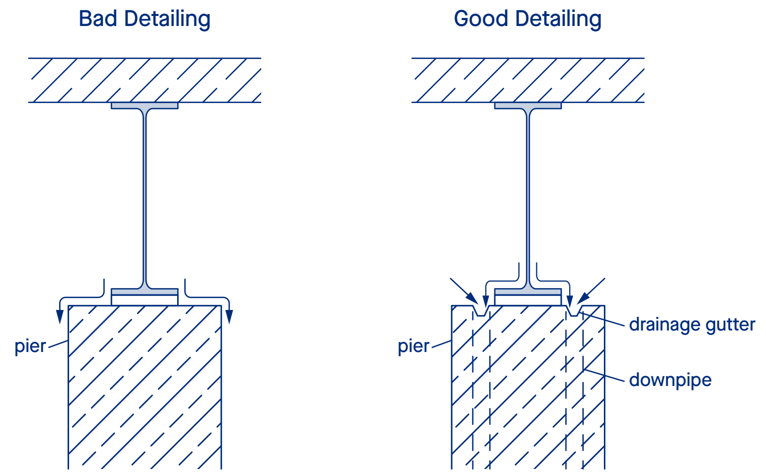

Figure 3-19: Drainage gutter on the top of the piers

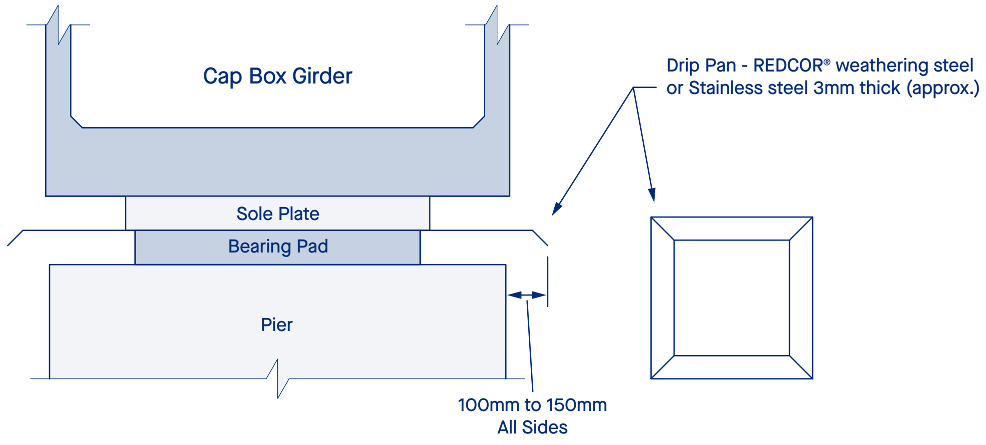

Figure 3-20: Well-designed drip pan installation

Figure 3-21: Drip pan example below box girder

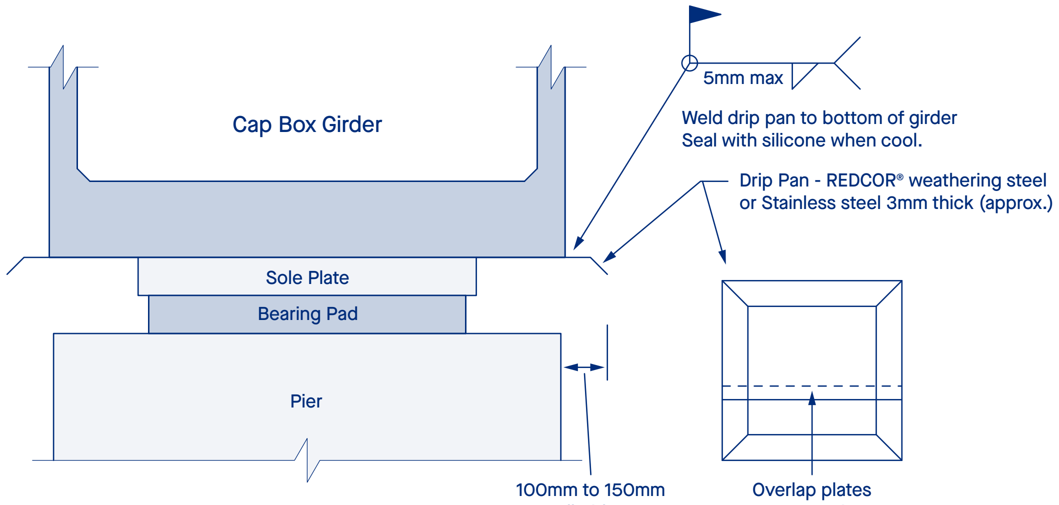

Figure 3-22: Design of retrofitted drip pan

(a) Concrete column without protection of drip pan

(b) Insufficient drip pan overhang permits run-off to blow onto pier

(c) Stain-free concrete due to installation of drip plate

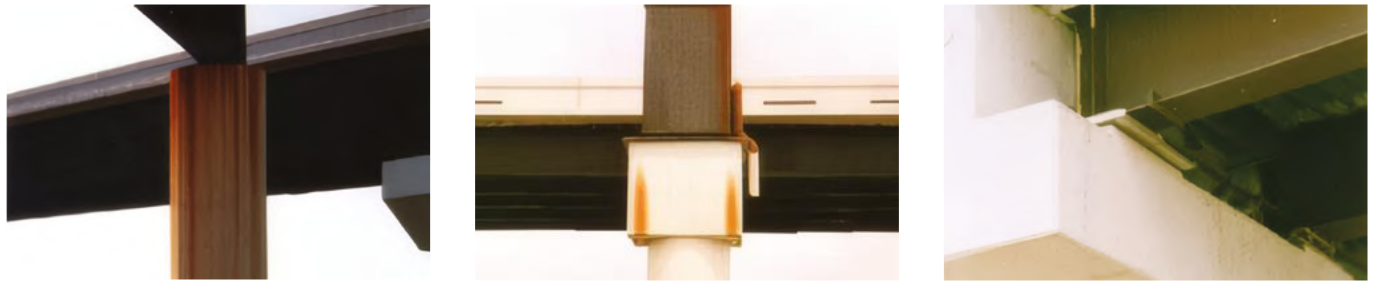

Figure 3-23: Good and bad detailing of a drip plate and drip pan – Images courtesy of TxDOT

Figure 3-24 shows good and bad detailing of the retrofitted drip pan.

(a) Improperly retrofitted drip pan on pier allows staining

(b) Poor welding on retrofitted drip pan

(c) Properly retrofitted drip pan on pier

Figure 3-24: Good and bad detailing of a retrofit drip pan – Images courtesy of TxDOT

It should always be kept in mind when designing a weathering steel bridge to make it easy to inspect. Inspection is required to ensure that an adherent protective patina has been formed and is not flaking off, that moisture and detritus are not collecting, and that the thickness of the structural elements is as expected by the Bridge Designer at that point in time. A programme of monitoring (in accordance with Section B2.2 of the Guidelines for Bridge Management (Austroads 2004) should be specified, and the design must make allowance for this to be done. All parts of the bridge should be designed to be readily accessible for an appropriate level of inspection.

3.8.5 Hanger Plate and Pin Connection

Hanger plate and pin connections (see Figure 3-25 for details) at cantilever expansion joints of girders are exposed to water leaking through open deck joints. If these details are of weathering steel they can be severely corroded in the gap between the girder web and the hanger plates. Also, there can be some galvanic corrosion of the steel girder with respect to the bronze washer installed between the girder web and the hanger plates. If the corrosive attack is severe, the gap between the girder web and hanger plates becomes tightly filled with rust, and may lead to pack out failure as discussed earlier. These details are bad from a corrosion standpoint even if the joints are painted.

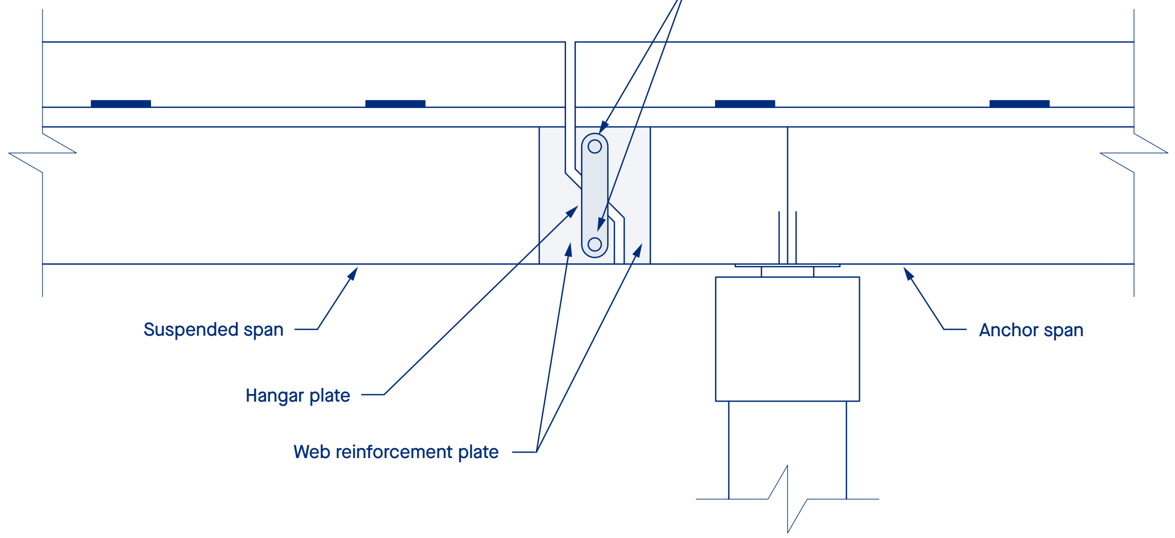

Figure 3-25: Hanger plate and pin connection

3.8.6 Box and Tubular Members

Box and tubular members for welded girders, columns and trusses constructed of weathering steel will corrode on the inside if water is continuously present. This can be prevented by fully sealing the member or ensuring adequate drainage holes are provided. If a pipe carrying water passes through a girder, drain holes should be provided in the bottom of the girder so as to drain water in the event of a leak.

Consideration can also be given to insulating the water line to prevent condensation on the pipe exterior. Drain pipes should not pass through box members.

3.8.7 Box Girders

Composite steel-concrete box girders cannot be hermetically sealed, because moisture may come in through cracks in the concrete deck slab. Therefore, provision must be made for drainage of moisture and for adequate ventilation to minimize corrosion of the interior.

To drain any water that may occur in a box girder, drainage runs of sufficient size through transverse diaphragms and internal transverse stiffeners should be provided (ECCS, 2021). Drainage downpipes routed through a box girder should be absolutely tight and air gaps between the box girder and the drainage structure should be avoided.

Wetting of the steel structure by water from (leaking) drainage pipes must be prevented. If there is a risk of blockage in the drainage pipes or drains due to leaves, rust or dirt, the smallest diameter of the drainage devices should not be less than 150 mm. Since drainage pipes do not always remain tight, especially at pipe joints and drainage openings, ensure that the pipes, pipe joints and cleaning openings have a sufficiently large distance from the steel structure. Condensation may also occur on drainage pipes. This has to be considered when routing uninsulated water pipes in box girders.

If the interior is accessible so that periodic corrosion inspections can be made, coating of the interior is unnecessary. However if future interior coating becomes necessary, it would be helpful to inspectors if a white alkyd coating system was used. If interior inspections to monitor condition of unsealed boxes are not possible, coating the interior is recommended.

Alternative steps to coating may be taken to insure satisfactory corrosion performance for the interior of unsealed, composite box girders. These are:

- Waterproof and pave the bridge deck to prevent water from leaking through the deck and into the box girder interior
- Provide access holes with locked covers on screens to keep out vermin and birds (see Figure 3-22) but to allow ventilation and inspection
- Drain the girder by drilling 50 mm diameter holes in the bottom flange at the lowest point of each compartment. Where there is <2 degrees slope to bottom flange provide drainage holes at ~10m centres. Insert a small tube fitted with an insect screen into the hole so that the top of the tube is flush with the top surface of the bottom flange and the tube itself protrudes below the flange
- Where longitudinal stiffeners are used on the bottom box girder plate, drill drain holes, as shown in Figure 3-26, from (AISC, 2022)
- Locate and size the inspection hatches for easy access by the inspector

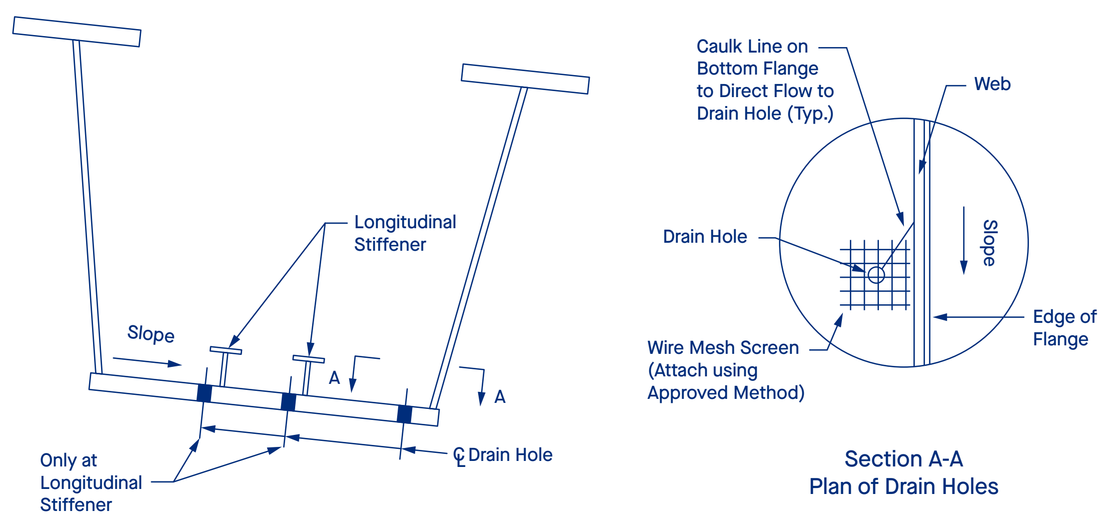

Figure 3-26: Example of drain holes and use of mesh screen

3.8.8 I-Girders

Corrosion of weathering steel members is most likely to occur on horizontal surfaces, in crevices and re-entrant comers. On I-girder bridges it has been found that locations where corrosion attack is most likely are the top surface of bottom flanges, gusset plates, longitudinal stiffeners, splices of horizontal and sloped members and where flanged gusset plates contact bearing and intermediate stiffeners. Therefore, to reiterate the guidance given above, good design and detailing practice should involve:

- Eliminating ponding, crevices and minimizing re-entrant comers
- Providing for good drainage at low points when girders slope away from the centre supports and toward the end supports
- Changing the flange thickness instead of the width, where welded flange splices are used, because changes in widths may cause uneven water flow
- Reduce the likelihood of bird nesting via the use of proprietary bird control solutions, such as bird spikes, netting, wire, and/or other non-lethal bird deterrents methods such as bird deterrent gels

3.9 Removal of Rust Stains

Although rust staining should not occur on a well detailed weathering steel bridge, it is worth noting that concrete, stone, wood, galvanised steel and unglazed brick are difficult to clean.

Hence, it is recommended that substructures are sealed with anti-graffiti or other washable organic coatings to facilitate cleaning with commercial products, should rust staining occur.

The following materials are subject to minimal staining and can generally be cleaned relatively easily:

- Ceramic tile and glazed brick
- Washable air-drying and thermosetting organic coatings
- Stainless steels
- Aluminium, anodised and un-anodised

The following materials are prone to severe staining and are difficult or impossible to clean:

- Concrete, cementitious plaster and stucco
- Galvanized steel
- Unglazed brick
- Stone
- Wood

Some of the graffiti removal procedures in Section 6.3 can also be used to help remove rust stains off certain materials i.e. For example, BMG Chemicals' Graffiti Red was tested and can remove rust stains on concrete & other materials very easily and should be checked with the supplier of the product.

When detailing, particular care should therefore be taken to ensure that rust-laden water will not come into contact with such materials.

An example of the staining which can result from run-off onto concrete compared to a better detailed example is shown in Figure 3-16 and 3-17.

Figure 3-27: Example of ring bark corrosion on weathering steel sculpture, embedded in the ground

3.10 Interface Protection

Other issues to consider include:

- Elements buried in soil should be coated, up to 100mm above the ground, with a minimum of 350microns of a suitable barrier coating, such as a high build epoxy or polyurea, to minimise non-atmospheric corrosion and risk of 'ring bark' corrosion. See Figure 3-27
- Interfaces between steel and concrete should be separated, using a minimum thickness of 150 microns of non-conductive and non-absorbent barrier material or coating
- Elements embedded in concrete need not be coated, except 100mm from below and 50mm above the interface
- Contact between steel and treated timber must be avoided, unless a mastic strip or damp proof course is placed between these components

3.11 Connection to Other Materials

In general, any connection to a galvanically dissimilar material runs the risk of galvanic corrosion of either the weathering steel or the other material (such as galvanized bolts). However, this will only occur in the presence of water. Therefore for connections between dissimilar metals inside a dry weathering steel box girder, the risk of galvanic corrosion is minimal.

Care should also be used if weathering steel and stainless steel (for example in bearings) are connected without appropriate electrical insulation. In this case the weathering steel would become anodic and corrode preferentially, although experience has shown that provided the joined area does not act as a crevice to attract water; serious problems are unlikely to arise. Such an application is the drip pan detail shown in Figure 3-18. The contact surfaces and regions around them can be coated with a 150 microns of non-conductive barrier coating.

If welding dissimilar metals, such as stainless steel to weathering steel is required, this should be undertaken in accordance with Appendix H of AS/NZS 1554.6.

Finally, for contact surfaces between weathering steel and stainless steel fasteners, for situations where weathering steel fasteners are not available, ECCS (ECCS, 2021) states that it is permissible; especially on vertical surfaces where water ponding is not possible. However, as per Clause 3.3.3.2 of AS/NZS 2312.1, it is good practice to separate dissimilar metals by electrically isolating them. This can be achieved by applying the above 150 microns of non-conductive barrier coating at the weathering steel surface /stainless steel interface or the use of non-conductive washers.

3.12 Further Protection – Coating

There are some weathering steel bridges in existence where the designer wished to provide further protection from the outset. Coating the steelwork has been specified in areas where it was thought that the environment would prevent the formation of the protective patina. This can either be due to the accumulation of debris, salt, and/or other contaminant. Such areas include; the top surfaces of bottom flanges, perhaps together with some of the bottom of the web, areas below deck joints and under the ballast in trough section railway bridges.

In such circumstances, coating these potential problem areas should be considered. There is some evidence that, although a barrier coat lasts no longer than on ordinary structural steel, if the coat is damaged or degraded the self-stifling properties of the rust which is formed helps to prevent under-creep. This can be beneficial since damaged areas which require touch-up and recoat will not spread to any extent even after a prolonged period of time.

Implications for the appearance of the bridge should be considered when specifying local coating and the colour of the coating system on exposed steel work should be selected to match the expected colour of the steel. These should be able to be applied on to a high pressure water cleaned surface and should comprise a single coat system where possible. See AS/NZS 2312.1, for guidance on a suitable coating system such as a direct to metal high build water based acrylic elastomer or a moisture cured urethane mastic.

It is recommended that a qualified coating specifier is used to prepare, or review, the refurbishment specification. The required qualification for such a specifier should be one of the following:

- AMPP (previously NACE) Protective Coating Specialist or Corrosion Specialist
- Australasian Corrosion Association (ACA) Technician or Technologist with successful completion of the ACA's Coating Selection and Specification Course and/or certified to AMPP Coating Inspection Program (CIP) Level 2

3.13 Stray Current Corrosion

Stray current corrosion is an often-misunderstood corrosion mechanism, that is commonly attributed when "unexpected" section loss is found, or in the case of protection coatings, premature coating failures, on a bridge superstructure.

Stray current corrosion can only occur in either buried or immersed portions of a bridge, typically in the bridge substructure; or on above ground steel components that are in contact with the ground through sleepers and ballast; when they are exposed to direct current sources from electric trams and trains.

To mitigate the risk of stray current corrosion, ensure the appropriate earthing and bonding design and detailing of the steel components (whether the superstructure, substructure and/or metallic components such as rails and reinforcement) is undertaken; in accordance to standards such as EN 50122-1, AS/NZS 3000, AS 2832.1 and documents such as Electrolysis from Stray DC Current (TfNSW, 2019).

Figure 3-28: Bidgee Bidgee Bridge, Rosehill, NSW
Made from REDCOR® weathering steel

4. Fabrication and Construction

4.1 Cold and Hot Forming

Cold forming can be applied to (hot-rolled) weathering steels to the same grade as for general structural steels. The bend should be applied with the axis transverse to the rolling direction.

As with most high strength structural steels, cold forming is not generally recommended for structural shapes or for plates and bars over approximately 20 mm thick, in accordance with AS 3678. The recommended minimum cold bending radii are given in AS3678 for plate thickness up to 20mm however, note the need to condition the plate edge prior to cold bending. Also, where fracture toughness is important, AS 4100, in the sections on brittle fracture, provides guidance on the effect of cold forming strain levels on the resultant change in notch toughness of structural steels.

When a member or component is subjected to an outer bend fibre strain exceeding 1.0% during fabrication, the permissible service temperature for each steel type should be increased in accordance with AS/NZS 5100.6 Section 14.5.3. This section applies to the weathering steel subject to cold bending to form L and C-shapes. The suitability of the cold-bend products for use at the minimum design service temperature should be checked.

Hot forming can affect both the tensile properties and the notch toughness, particularly in steels without sufficient grain refining elements. Even in steels with grain refining elements (including most weathering steels) certain supply conditions (such as thermo- mechanically controlled rolled) may incur a significant loss of strength from hot forming. Steel supplied in the normalised condition may be able to sustain some heat treatment or hot-forming processes, if the steel is heated to the appropriate temperature range.

Accordingly, a maximum temperature of 600°C is recommended; see Clause 6.6.5 of AS/NZS 5131 or contact the steel manufacturer for guidance.

4.2 Cutting

Flame-cutting (for example oxy-acetylene or oxy- propane) or plasma-arc cutting of weathering steels can be carried out using the same procedures as would be applied to carbon steels of similar carbon equivalent value and thickness.

Application of preheat temperatures similar to those used for welding will avoid excessive hardening of the flame-cut edges. If required, grind the hardened edges, as if left, this will affect the formation of the protective patina at that surface.

Any cut surface not incorporated in a weld should have a roughness not greater than the appropriate value given in Table 6.5 of AS/NZS 5100.6 as applicable for the fatigue detail category specified.

As general guidance, for a thermally cut edge not subsequently fully incorporated in a weld, cracking is more likely to occur if the hardness exceeds 350 HV (Vickers Pyramid Number).

4.3 Welding

4.3.1 Welding Processes

Weathering steel can be welded using the same techniques used for low alloy steels. Typical welding processes used are submerged arc welding (SAW), flux-cored arc welding (FCAW), gas metal-arc welding (GMAW or MIG) and manual metal-arc welding (MMAW).

4.3.2 Preheat

The welding of structural weathering steels is similar to that of conventional structural steels, but weathering steels generally have a higher Carbon Equivalent (CE). Steel grades CW300, HW350 and WR350 have a Weldability Group Number 5 in accordance with the Table 5.3.4(A) of AS/NZS 1554.1. It is based on the typical maximum carbon equivalent encountered in Australia and New Zealand weathering steels, rather than a maximum specification limit normally applied. The preheat temperature should be determined in accordance with Section 5.3 of AS/NZS 1554.1.

4.3.3 Hot Cracking

As weathering steels typically contain levels of phosphorus and/or copper significantly higher than that found in C-Mn structural steels, in certain joint configurations and higher heat inputs typically associated with the SAW process, the weld may be at risk of hot cracking; also referred to as solidification cracking. The solidification crack susceptibility of weld metal is affected by both its composition and weld-run geometry (depth/width ratio). The chemical composition of weld metal is determined by the composition of the filler material and the parent metal, and the degree of dilution. The degree of dilution, as well as weld-run geometry, both depend on the joint geometry (angle of bevel, root face and gap) and the welding parameters (current and voltage). At normal heat inputs used in manual or mechanised welding the risk of hot cracking is considered low, however at higher heat inputs, particularly >2.5kJ/mm, and for fillet weld runs having a depth/width ratio greater than 1, the risk of hot cracking is relatively high. If in doubt, a hot cracking test should be considered as part of the weld procedure qualification test requirements to verify freedom from hot cracking. Test methods are provided in AS 2205.9.1.

4.3.4 Selection of Welding Consumables

Welding consumables (electrodes) should be selected by AS/NZS 1554.1, Section 4.6.1.2 and Table 4.6.2(C). The specific recommendations applicable to the above AS/NZS steel grades and welding techniques are:

a) For single-run fillet welds and butt welds made with a single run or a single run on each side made with or without where weaving, weathering steel welding consumables should be selected from Table 4.6.1(C) of AS/NZS 1554.1.

b) For capping runs on multi-run fillet or butt welds, welding consumables should be selected in accordance with Table 4.6.1(C) of AS/NZS 1554.1. This also applies to the last filler run(s) if the weld reinforcement is to be removed. If the ends of the welds are exposed, the use of weathering steel consumables may be required throughout the full thickness of the joint. For internal runs, welding consumables can be selected in accordance with Table 4.6.1(A) of AS/NZS 1554.1

c) Hydrogen-controlled welding consumables should be used for all welds.

d) Commonly used low alloy filler metals for weathering steel applications include those with a minimum nominal nickel content of one percent. That alloy content is sufficient to provide atmospheric corrosion resistance similar to the weathering steel, and the cost is typically less than other low alloy filler metals with acceptable properties. Filler metals with a higher nickel content and other alloying elements, can be used in accordance with the Table 4.6.1(C), AS/NZS 1554.1. Weathering steel welding consumables with a higher impact grading are also acceptable.The use of higher Ni-bearing welding consumables with Ni content exceeding 1% (e.g., 2% Ni) is a conservative approach to ensure patina formation, and is aligned with EN 1090-2 requirements

e) Wherever possible, a designer should specify fillet welds that can be made in a single pass.

Welding of different weathering steel grades to each other is possible and permissible. Weathering steel can also be welded to other weldable, non-weather-resistant structural steels like non-alloyed structural steel. In this case, the weld seam zones as well as the adjacent components made of non-weathering steel must be protected (painted) according to their required corrosion protection.

Welding of the weathering steels to stainless steels should be to AS/NZS 1554 Part 6. It also includes the selection of welding consumables. The use of austenitic filler material of type 309L will be normally required. The use of carbon steel or weathering steel consumables may lead to embrittlement and loss of ductility in the weld. Following welding, the weld area must have heat tint removed. If practical, the welded area should be painted 50 mm beyond the weld line on both carbon and stainless steel parts of the joint to prevent galvanic corrosion. Do not use a sacrificial primer (such as a metallic zinc-pigmented formulation) on the stainless steel as any penetration of water through the topcoat will cause galvanic effects between the zinc and the underlying stainless steel. This reaction would blister the topcoat. If there is no organic coating applied, then the stainless steel nearest to the weld will probably suffer rust staining.

The American Welding Society's Standard Table 3.3 of AWS D1.1 provides a broader range of consumable options for common welding processes where a matching patina is required on multi-pass welds in particular. Consumables meeting these requirements may be used to weld WR350 and HW350 grades.

4.3.5 Quality Requirements

All welding fabrication including quality management and control, and inspection should comply with the Construction Category 2, 3 or 4 of AS/NZS 5131 as specified for the project and the relevant part of AS/NZS 1554 Part 1 or Part 5 as well with the additional requirements for the Fatigue Detail Categories of AS/NZS 5100.6. The fabricator should demonstrate the ability to produce sound welds via a documented weld procedure and welder qualification tests. The fabricators should ensure that all welding and related activities are managed under a suitable quality management system. Such a system should generally comply with the requirements of AS/NZS ISO 3834 part 2 or 3.

The inspection requirements for welded connections can be found in Section 13 of AS/NZS 5131. The Recommended extent of Non-Destructive Examination (NDE) can be found in Table 13.6.2.2(A) of AS NZS 5131. The extent of testing depends on the Construction Category (CC) specified for the structure/component and the weld type.

The ultrasonic testing (UT) is typically specified for the full penetration butt welds only. The application of UT to fillet and partial penetration compound welds is usually restricted to the examination to exclude potential weld defects on a case-by-case basis. In the production situation, the inspection of fillet and partial penetration compound welds should be based on the procedural control of the correct application of the welding procedure including inspection of the joint preparation before welding, inspection during welding and the visual scanning of welds after welding.

4.4 Surface Preparation

Further quality requirements are given in the Australian Standard for Fabrication and Erection Standard for Structural Steelwork, AS/NZS 5131. It is recommended this Australian Standard be consulted for guidance on the welding fabrication of weathering steel.

4.4.1 Mill Scale Removal

Abrasive blast to Commercial Blast Cleaning cleanliness standard as defined under SSPC SP-6/NACE No 3 (similar to ISO 8501-1 Sa 2), which removes oil, grease, dirt, rust scale and foreign matter. It also removes most of the mill scale, coating, and rust in the bottom of pits except for slight streaks or discolorations. At least two-thirds of the surface area should be free of visible residues except for the discolorations previously mentioned.

The resultant surface will weather relatively uniformly. The process is considerably less expensive than near-white metal blast cleaning (SSPC-SP10/NACE No 2, similar to Sa 2½); however the finished appearance is less uniform during the first 6 to 12 months, after which the finish will become more uniform over time.

It is important that all contaminants are removed from the surface of the steel to enable it to form a uniform protective rust patina; as shown in Figure 4-1. Mill scale will be undercut during the weathering process and will fall off eventually, but will also delay the formation of a uniformly coloured protective layer; hence it is recommended that mill scale is removed from the whole surface and not just the faying surfaces (Section 3.5.2). As such the following clause should be included in the bridge specification.

After fabrication and prior to erection, all weathering steel components shall be abrasive blast cleaned to SSPC-SP 6/ NACE No 3 to remove mill scale and other contaminants. This shall be immediately followed by a minimum of 3 cycles of wetting using potable water and drying, ensuring that the steel surface is fully dry before wetting again, to assist in the formation of the protective patina and provide a uniform finish.

Figure 4-1: Example of surface contamination in the form of mill scale (above) and blasting of the steel surface (below)

4.4.2 Grinding and Other Cleaning Methods

Prior to abrasive blast cleaning, or if the following contamination was caused during erection, the following surface preparation methods should be undertaken.

- Wax-based crayons should not be used to mark weathering steel, see Figure 4-2. However, if such marks are present, they should be removed by degreasing with a suitable solvent
- Oil, grease, and cutting compounds may also be removed by solvent degreasing (AS 1627.1 or SSPC-SP1). Alkaline cleaners or a combination of detergents and steam may also be used. When alkaline cleaners and detergents are used, their use should be followed with high pressure water cleaning to remove any residue
- Acids should not be used for cleaning because of the possibility of acid residues remaining on the steel surfaces and causing corrosion
- Localized weld spatter or other welding residues may be removed by power tool cleaning (SSPC-SP 15)
- Loose deposits of rust, rust scale, coating or other foreign matter may be removed by hand tool cleaning (St2 or SSPC-SP 2) such as scraping or wire brushing. More adherent deposits may require power tool cleaning (St2 or SSPC-SP 3) or brush-off blast cleaning (Sa1 or SSPC-SP 7/NACE No 4)

The above surface preparation processes are all described in AS 2312.1, and defined in the relevant SSPC/ NACE documents.

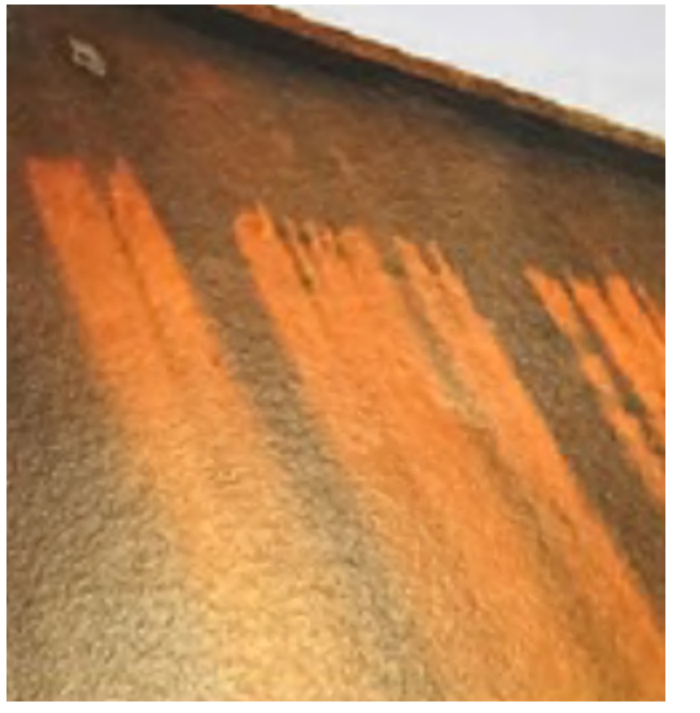

Figure 4-2: Staining from surface markings, not correctly removed

4.5 Storage, Handling and Erection

Storage of weathering steel sections and plates should ensure that the protective rust patina continues to develop following preparation of the surface, as discussed above. This means that, in ideal conditions, the steel will be stored such that each surface is alternately wetted by rain; or preferably hosed down daily for one week, and dried naturally after every wetting. Particular care must be taken to ensure that plates and sections are not stored so that they become permanently wet, or entrap moisture or dirt. This may easily occur, for example, if a plate is not fully supported so that it sags and thus provides a water collecting area.

Covering with plastic or tarpaulins is not recommended as it promotes condensation and prevents the alternate wetting and drying. Supports should not be from chemically treated timber or use absorbent material.

Contamination of the surface should be avoided. This may arise from concrete, mortar, asphalt, coating, oil or grease. In particular, marking the surface for reference during fabrication with wax crayons should be avoided, since this marking can be very difficult to remove. Consider hard stamping for identification of members or joints.

The use of metal slings for handling should be carefully controlled, since they can damage the developing surface protection layer on the steel. While this will eventually redevelop over time, it will give an uneven appearance until removed by weathering.

During erection, continue to protect the sections from contamination and damage. Site welded joints may require special treatment, such as grinding off excess weld on upper surfaces of flanges to avoid potential corrosion traps, and spot abrasive blast cleaning of these and support areas, to ensure that all surfaces weather to a uniform colour in a similar period of time.

4.6 Final Site Cleaning

Where care has been taken in handling, storage and erection, it may be possible to avoid any final site cleaning. However, if contaminants have been allowed to accumulate, they must be removed, either by washing, by chemical means, or by a site blast clean. Similarly, areas where severe physical damage has occurred may also require blast cleaning after any repair (such as heat straightening).

4.7 Protection of Piers and Abutments

If there is any risk of piers and abutments being stained by rust laden water run-off during erection, consideration should be given to providing temporary protection by wrapping them with polyethylene sheeting or its equivalent. This sheeting should remain in place and be kept free of damage until the final construction inspection is made. See Section 3.8 for detailing for stain prevention.

In the case that the sub-structure develops stains, they may be removed by abrasive blasting, or with a commercial cleaning solution after completion of construction.

4.8 Guard Rails and Light Poles

Guardrails and light poles on weathering steel bridges should, where possible, be connected to the concrete deck rather than directly to the supporting steel beams. In most instances these will be galvanized steelwork, with an additional barrier coating or sealant to minimise the risk of crevice corrosion, see Section 3.11.

Figure 4-3: Fitzroy River Bridge featuring welded beams made from REDCOR® weathering steel, Kimberley, WA

5. In-service Inspection

5.1 Requirements for Inspection of Weathering Steel Bridges

All bridges, in whatever material, require periodic inspection to confirm that they are performing satisfactorily and to identify and mitigate defects at the first opportunity. A weathering steel bridge, properly designed and detailed, and in the correct environment, should deliver trouble free performance. However, regular inspection of the bridge structure will assist in the early detection of potential issues, and their prompt remediation will assist to minimise the risk of more significant problems in the future.

All parts of the bridge should therefore be designed to be readily accessible for an appropriate level of inspection.

5.2 Level 1 Inspections (Routine)

Level 1 Inspections (Routine) of weathering steel bridges should be carried out by a suitably trained bridge inspector, as described in Table 5 of (Austroads 2004). The surface condition of the protective rust patina is a good indicator of performance. An adherent fine grained rust patina indicates that corrosion is progressing at an acceptable rate, whereas coarse laminated rust layers and flaking suggests unacceptable performance. Other signs to look for and areas to investigate during visual inspection include:

- Leaking expansion joints and trough drain holes
- Accumulation of dirt or debris
- Moisture retention due to overgrown vegetation or birds nests
- Faulty drainage systems
- Condition of sealants at concrete/steel interfaces
- Excessive corrosion products at bolted joints (“pack rust”)
- Signs of surface contamination, such as graffiti, guano or other material
- Signs of water ponding
- Signs of localised change to the protective patina layer formation, especially relating to loosely adhering flaky corrosion, that could indicate a leaky deck and/or joint, or other causes of permanent moisture

If any issues are noted during these inspections, the cause should be identified and the problem rectified as soon as possible.

5.3 Level 2 Inspections (Condition Assessment)

This level of inspection should be undertaken every 3 years by an accredited inspector, as described in Table 5 of (Austroads 2004).

Provided that provisions have been made for it in the design, the routine inspection described above will pose no particular difficulties.

However, the more detailed inspection of weathering steel bridges differs in a number of respects from, and in general is more difficult than, the inspection of coated carbon steel bridges.

One of the advantages of weathering steel is that the surface can be seen directly. However, whilst a heavily corroded surface will be obvious, an inspector must be familiar with the various colours (see following section), textures and general appearance that the rust patina can assume when exposed to different environments in order to judge whether or not the patina is acting in a protective manner. Furthermore, visual appearance on its own may be unreliable and mechanical or other tests may be necessary to determine whether or not the film adheres to the underlying steel base.

One problem which may arise with many such tests (for example wire brushing, or preparation of the surface for ultrasonic investigation) is that the appearance of the protective film may be changed; it will take time for this to return to a uniform appearance.

Whilst design against fatigue should ensure that cracking does not occur during the service life of the bridge, detailed inspection is required to confirm that this is so. As described below, detection of fatigue cracks in uncoated weathering steel bridges can be more difficult than detection of cracks in coated steelwork.

Other signs to look for and areas to investigate during the condition assessment include:

- Condition of the protective rust patina on near flat surfaces or corners
- Condition of the protective rust patina near welded details
- Bolted joints, especially with regard to crevice corrosion and loose bolts

5.4 Surface Appearance

An inspector must be able to distinguish between a protective and a non-protective rust coating. Normally this can only be done at close range (within 1 metre distance). The appearance will give the first indication of the quality of the protective film. Whilst only experience can make an inspector competent in such matters, some guidelines are given below.

5.4.1 Adherence

The protective layer should be tightly adhering. This metric is arguably the most reliable for determining the weathering steel performance, as discussed in (AISC, 2022). Adherence can be confirmed by no change in surface condition occurring due to pounding with a rubber mallet or the inability of the patina to be rubbed off or pried loose by dull hand tools (e.g. putty knife). Examples of the surface texture of UWS meeting this criterion are shown in Figure 5-1.

(a) (b) (c)

Figure 5-1: Examples of good performing patina

In contrast, Figure 5-2 shows examples of easily disturbed patinas. The patina in Figure 5-2 (a) can be scraped loose with a dull putty knife. Figure 5-2 (b) shows a patina that readily crushed under light impact. Figure 5-2 (c) shows a patina that can be pried loose with a fingernail.

(a) (b) (c)

Figure 5-2: Examples of poorly performing patina

There are two exceptions to the above comments on adherence being indicative of performance. One is that in the first few years of service or in very benign environments where patina development occurs slowly, fine particles that are easily removed from the surface are not of concern (see comments below on texture). The second is that if the mill scale is not removed, as discussed in Section 4.4.1, then the mill scale will likely be easily removed from the surface. It is because of the difficulty in distinguishing between mill scale and base metal corrosion for inspectors without significant experience that removal of the mill scale prior to erection is recommended, as well as achieving a more uniform surface finish, as discussed earlier.

If wire brushing is used to evaluate the weathering and aesthetics are of importance, this should be done at locations not viewed by the public, as the surface removal will leave a lighter colour (that will re-darken with time).

5.4.2 Texture

The size of the particles forming the protective patina often directly correlates to adherence, with larger particles being less adherent and more indicative of corrosion concerns. Such texture is often difficult to assess without adequate proximity to the surface, and thus such an inspection is essential for weathering steel structural members (see Figure 5-4).

Rust particle sizes of 3mm or less are not of concern. Such particles sizes result in relatively smooth surfaces, manifesting in many different appearances as shown at close range in Figure 5-1. Granular rust flakes exceeding 6mm diameter are possible indications of a non-protective patina see Figure 5-2 (a). While, sheet-like layers of rust (see Figure 5-2 (b) and (c)) are clear indications of a non-protective patina. Also shown in Figure 5-3, an example of granular rust flakes, taken in Bowen, Queensland.

Figure 5-3: Example of granular rust flakes

This highlights the importance of the inspector being sufficiently close to ascertain the texture of the patina, as without access for close-range inspection, nearly all weathering steel surface will appear to have a smooth texture.

The Japanese have developed a 5-step classification of surface appearance using a photographic ‘appearance index’ (Kihira 2004) and have attempted to use this as method of estimating corrosion rates (Hara 2007).

5.4.3 Colour

The protective rust coating colour should begin as yellow orange after the initial stage of exposure, becoming light brown, and finally chocolate to purple brown (ranging between AS 2700 Colour reference R65 Maroon, and Colour reference X64 Chocolate). See Figure 5-4 for an example of 6 months old patina (left) and a fully formed patina after 50 years (right). Note that in some lighting conditions its appearance can vary from metallic grey to purple.

The timing of the colour and texture changes can vary with atmospheric conditions and the degree of direct exposure to rain and sunlight; as steelwork shaded from direct sunlight will remain damp for longer compared to sunlit surfaces, hence delaying the patina formation on those surfaces. A rural, unpolluted atmosphere (typical of most rural areas of Australia, Corrosion Category C2 regions, as discussed in Section 2.3.1) or sheltered interior beams, will result in a lighter colour and dusty texture, taking significantly longer to change, potentially up to 16 years (or longer). The steel composition can also affect this – the greater the extent of alloying elements, the darker the final colour.

In New Zealand, it has been identified that the protective patina may take up to 8 years to fully form in ideal conditions; this may increase up to 16 years (or more) in areas with higher time of wetness. During this period staining from rust run-off reduces as the patina fully forms during many cycles of wetting and drying (see Section 3.8.4 for ways to address run-off).

If the condition cannot be reliably ascertained, it may be necessary to remove part of the protective layer to determine the extent of pitting and to measure the section loss. However, note the limitations with measuring the steel thickness given below.

It should be noted, that in polluted and/or high salinity environments, the darker colour that implies the patina is formed, may develop within a shorter time frame (from 2 to 8 years). However, evidence has shown (Morcillo et al 2013) that this does not mean that the protective patina has fully formed. Therefore, monitoring the texture of the surface and measuring the steel thickness will assist in determining when the patina has achieved its protective properties.

Figure 5-4: Example of a 6 month old patina (left) on Fitzroy River Bridge, WA, and a fully formed patina, after 50 years of service (right) on Frankland River Bridge, Tasmania

5.5 Measuring the Steel Thickness

While the condition of the protective rust patina should be regularly monitored (as discussed above), it is recommended that the measurements of the corrosion rate should be undertaken every 6 years in a C3 environment or every 12 years in C2. This is done by measuring the remaining steel thickness at clearly identified points of the structure. These reference points should be defined on the as-built drawings, or in the bridge maintenance manual, along with the original (reference) thickness measurements taken at the end of the construction period, using callipers and/or the use of an ultrasonic thickness gauge. Measuring internal surfaces is quite difficult using mechanical means.

If over time, it is identified that the corrosion rate is higher than the rate the corrosion allowance was originally based upon, then remedial measures may need to be considered. Note that the corrosion rate is usually higher during the first 10 years of exposure, after which the lower steady state corrosion is reached.

Therefore a minimum of 20 years of data is required, unless significant unexpected section loss during the period is identified.

Portable ultrasonic thickness gauges are available to measure the actual steel thickness. However it may be challenging to get an accurate reading, if the patina is still forming and the surface is still rough and easy to remove by hand. Because of this, it is also recommended that removable weathering steel coupons are installed to more accurately monitor the patina formation and measure the corrosion rate.

Allowance should be made for the installation of a minimum of two sets of test coupons on weathering steel bridges, on the primary structural members. The coupons should be cut from the same weathering steel plate used in the bridge, with the same surface preparation (as discussed in Section 4.4).

A total of 30 coupons, cut to 150 × 100 × 5mm (or the thinnest plate available for that bridge), to be installed per bridge, of which 15 are installed on the outer girder, that is exposed to the prevailing weather, and 15 on an inward facing girder surface, that is sheltered from the sun and rain washing. A set of three coupons per side could then be removed and tested on years 6, 12, 18, 24, and 36, to monitor the patina formation and confirm the section loss (and corrosion rate) by testing to ASTM G1-03(2011).

5.6 Detection of Fatigue Cracks

In a coated carbon steel bridge, the first indication of a fatigue crack is often the colour contrast between the coated surface and the rust stain in the vicinity of the crack. Such obvious signs will be absent in weathering steels; indeed, observations of crack growth in fatigue tests of weathered steel beams has shown that fatigue cracks less than 150mm long are very difficult to find by visual inspection. In actual bridges, the shortest crack that can be detected is likely to be even longer, since the crack forms a crevice which completely fills with rust during the service exposure.

The best course of action is to have a Welding Inspector, qualified to Clause 7.2 of AS.NZS 1554.1, conduct a visual inspection for cracks in potential locations, followed by a magnetic particle inspection for confirmation if that location contains a suspected crack.

Further detection using ultrasonic testing is recommended on those locations, with the removal of the rust patina for an accurate testing, where the ultrasonic testing will quantify the size and extent of the crack. For a short surface crack, it is essential to select the ultrasonic transducer with a higher frequency.

After the proper maintenance has been performed, the removed protective rust patina at the location will have to reform to regain its corrosion resistance properties. This is purely an aesthetic problem only, as once the layer has reformed the weathering steel will perform as expected.

Alternative testing methods such as Eddy Current testing (ET), Electromagnetic Acoustic Transducer (EMAT), and Magnetic Metal Memory Method (MMM method) can be considered to detect a crack in unpainted weathering steel. Further details can be found in (Salmen, 2018).

The application of the above non-destructive testing methods requires the qualification of a corresponding testing procedure to demonstrate the capability of the method to detect a (surface breaking) crack under the application conditions considered.

6. Maintenance

6.1 General

Routine maintenance of weathering steel bridges consists primarily in ensuring that the bridges are performing satisfactorily, and that they will continue to do so. It may include routine and/or minor remedial works as listed below; major works are described later under rehabilitation.

Such maintenance activities are usually undertaken based on the inspection findings, as discussed in Section 5.

Highway bridges, by their nature and use, accumulate debris; they become wet from condensation, leaky joints and traffic spray, and are exposed to salts and atmospheric pollutants.

Different combinations of these factors may create exposure conditions under which weathering steel may not form a protective rust coating and, for the continued satisfactory performance of the bridge, maintenance must be directed to preventing or rectifying such conditions.

6.2 Maintenance Procedures

The following examples illustrate the maintenance procedures which may be required, depending on the results of inspection:

- Remove loose debris with a jet of compressed air or with vacuum cleaning equipment
- Remove any poorly adhering layers of rust after removal of their cause (eg bird nests)
- Remove wet debris and aggressive agents from the steel surfaces by high pressure hosing. This is particularly important where the surfaces are contaminated with salt
- Trace leaks to their sources (on a rainy day or by hosing the deck near expansion joints and observing the flow of water). Repair all leaky joints
- Clean drains and downpipes
- Remove vegetation from the vicinity of the bridge
- Repair or reseal any leaking deck joints
- If necessary, install new drainage systems to divert water from superstructure and substructure
- In the event of "pack-out" of crevices at bolted joints, then the edges of the joint should be sealed with an appropriate sealant
- To prevent bird nesting on the bottom flange you can install piano wire as shown in Appendix D

6.3 Graffiti Removal

As with other forms of uncoated construction, such as reinforced or pre-stressed concrete, the removal of graffiti from weathering steel bridges is difficult, so measures to discourage public access to the girders should be considered.

However, this should be balanced with the need to provide access for inspection, monitoring and cleaning.

There are different methods to address the issue related to graffiti, the following options can be considered:

- Detail an anti-climb plate onto the ends of the bottom flange where the beam is accessible from the abutment, see Figure 6-1
- Use Graffiti Red product from BMG Chemicals Pty Ltd where you apply Graffiti Red liberally with a brush onto the graffiti 2 times in 3 minutes then rinse with water from a pressure nozzle. Full details in the Appendix C
- Apply an anti-graffiti coating at problem areas on the bridge, usually around the abutments. However, this will prevent the patina formation in those areas
- Apply a citrus based cleaner to the graffiti, with 24 hours of its application, before it is fully cured. Then low pressure water clean at 4000 psi
- Apply an alkaline poultice (e.g. Peel Away) to dissolve the graffiti and then steam clean followed by rinsing. Trials carried out for UK's Network Rail (Network Rail, 2016) found this method will result in the least damage to the protective patina and minimize local changes to the surface appearance
- Fully remove the graffiti and underlying protective patina layer, with high pressure water jetting at 10,000 psi. This option will fully remove the patina layer as well, resulting in a patchy look until it is reformed
- Use dry ice for removing graffiti, see (Brush, 2010)
- Leave the graffiti if not objectionable, as it will eventually be absorbed into the patina as it forms

Figure 6-1: Example of an anti-climb device

7. Rehabilitation of Weathering Steel Bridges

7.1 General

When a weathering steel bridge has corroded to an extent that further deterioration cannot be prevented by the simple maintenance procedures described earlier, rehabilitation may be required. Bridges designed, detailed and constructed in accordance with the guidelines given in this publication should not reach this stage unless circumstances beyond the control of the original design arise (for example a new industrial complex causing severe pollution is built close by). However, there are a few existing weathering steel bridges, in the Northern Hemisphere, where performance has been less than ideal, probably because some of the guidelines were not appreciated at the time of design and construction. This section is therefore also intended to assist those responsible for the rehabilitation of such bridges.

Rehabilitation may involve sealing of crevices, removal of poorly formed patina, strengthening and possibly coating of the corroded weathering steel. An alternative which has occasionally been used is the enclosure of the whole structure, although this is only likely to be economically viable in very unusual circumstances.

7.2 Sealing of Crevices

Since corrosion in crevices can be one of the major problems related to section loss, rehabilitation of such areas is a necessary preliminary to other work. Crevices can be treated as described below, depending on the type of detail and the degree of corrosion:

- For non-critical connections, such as at bolted restraints or bolted brackets, if practicable disassemble the connection, prepare the surface to a minimum of SSPC-SP 10/NACE No 2 (comparable to Sa 2½), apply a suitable coating, such as a non-conductive barrier coating, and reassemble. Alternatively;
- For critical connections, such as splices at the main girders, or other connections that cannot be disassembled, apply a penetrating sealer (such as a low viscosity epoxy, moisture cured urethane, or high ratio calcium sulfonate primer) to displace any water, caulk all edges with a moisture-cured polyurethane sealant, then stripe and coat the connection (lapping 50mm on the surrounding steelwork) with a compatible coating

7.3 Strengthening

Similar to carbon steel, weathering steel can be easily strengthened by simply bolting or welding new weathering steel plates, to the areas of the bridge that require strengthening.

The guidance given in Sections 3 and 4 relating to the detailing, fabrication and installation of the strengthening plates should be followed.

7.4 Use of Protective Coatings

The coating of a weathering steel bridge has the same issues and challenges as a carbon steel bridge. In both cases, preparing the surface for coating is essential. The suitability of abrasive blasting, either dry or wet, low pressure water cleaning, or high-pressure water jetting is dependent on the environmental constraints, accessibility and logistics for undertaking the work.

As part of preparing the surface, the refurbishment methodology should ensure the removal of salt and other contaminates, especially in deep pits. This includes choosing the appropriate coating system, based on the bridge condition, which should be undertaken in accordance to AS 2312.1.

7.5 Inspection and Maintenance of Coated Weathering Steel

Inspection of coated weathering steel is generally similar to that of coated carbon steel bridge, although the exact symptoms of breakdown may differ. It is recommended that an AMPP/NACE CIP Level 2 (or higher) coatings inspector is employed to undertake the inspection of a coated weathering steel bridge.

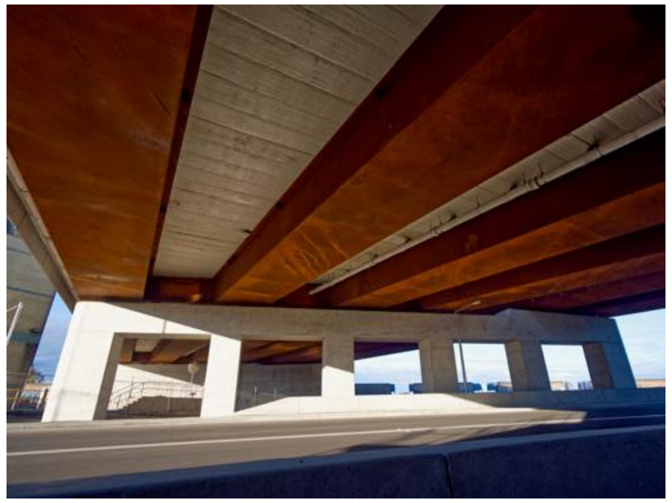

**Figure 7-1: Sydney Gateway Project Bridge SB21, Mascot, NSW
Box girders made from REDCOR® weathering steel**

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9. Appendices

9.1 Appendix A: Determination of Site-Specific Atmospheric Corrosivity Category

The following examples, outline the assessment methodology for determine the suitability of using weathering steel for the given site, as given in Section 2.3 herein.

Example A-1: Bridge located at Parramatta, NSW.

A bridge is to be built in Parramatta, within 10km from Sydney Harbour, and the prevailing wind is a North Westerly, i.e. blowing from the site toward sea; as shown in Figure A-4 as shown in AS 4312:2019.

Figure A-4 shows the site is on the C2/C3 boundary and greater than the 5km from the breaking surf, so weathering steel can be used in this site, with the corrosion allowance taken as 1.5 mm, as discussed in Section 3.3.

Example A-2: Bridge located at Whyalla, SA.

A bridge is to be built in Whyalla, within 1 km from the Spencer Gulf, and the prevailing wind is a South Easterly, i.e. blowing from the sea toward the site, as shown in Figure A-3 in AS 4312:2019; but the site is sheltered by the surrounding buildings.

As the site is in a C2 (Low) corrosivity environment and greater than the 500m from seawater limit, weathering steel can be used in this site, with the corrosion allowance taken as 1.0 mm, as discussed in Section 3.3.

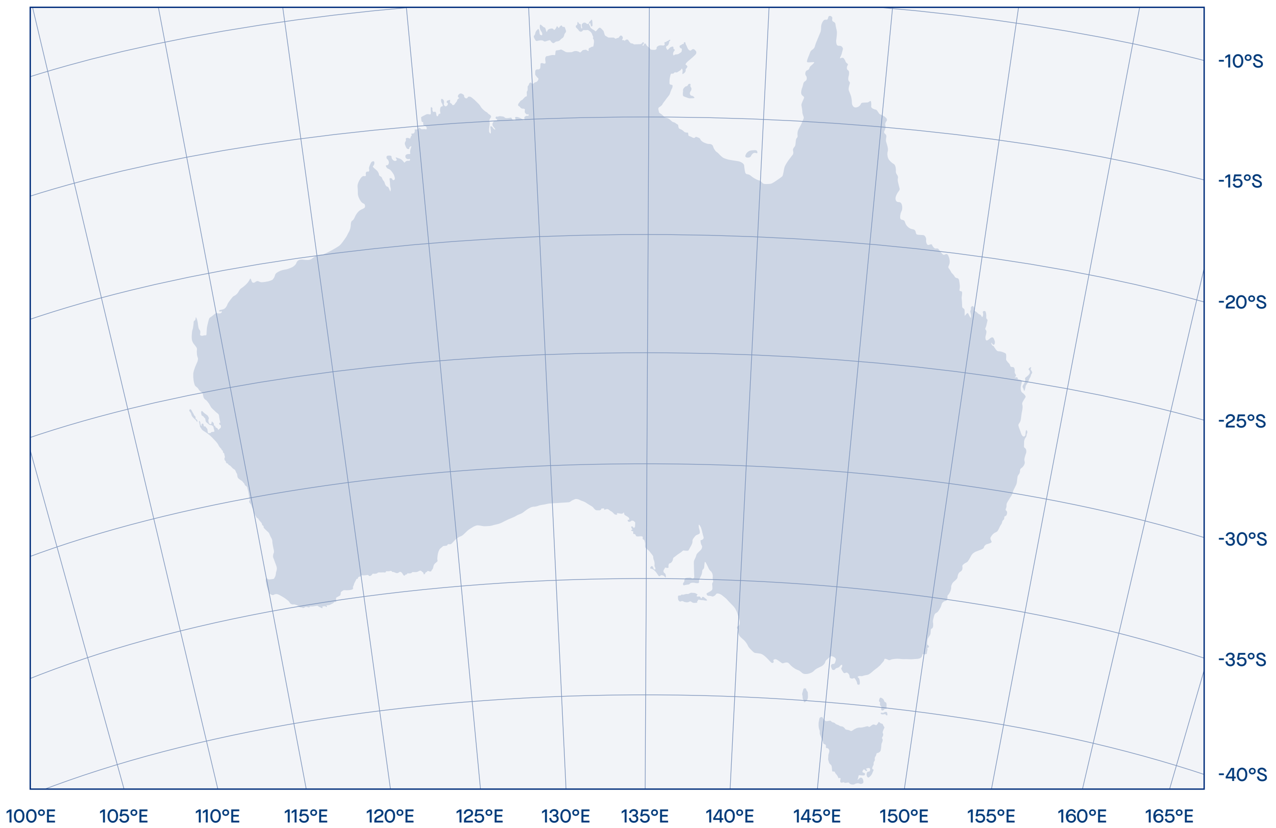
Example A-3: Bridge located at Yorke Peninsula South, SA.

A bridge is to be built in Yorke Peninsula South, within 3 km from the Spencer Gulf, and the prevailing wind is a South Westerly, i.e. blowing from the sea toward the site, as shown in Figure A-3 in AS 4312:2019 shows this region is C4/C5/CX. Alternatively, the site is within 5km of surf, so weathering steel cannot be used.

Example A-4: Bridge located at Learmonth, WA.

A bridge is to be built in Learmonth, within 3 km from the Exmouth Gulf, and the prevailing wind is a South-south Westerly, i.e. blowing from the site toward the sea.

Table A-1 of AS 4312:2019 shows the site is in a C2 (Low) corrosivity environment and beyond the 500m from the seawater limit, so weathering steel can be used in this site, with the corrosion allowance taken as 1.0 mm, as discussed in Section 3.3.



9.2 Appendix B: Australian Time of Wetness Table

Time of Wetness (TOW) calculation based on temperature and humidity data (5 years) from the Bureau of Meteorology (BOM). When using weathering steel the TOW needs to be below 80% for more than 40% of the year.

Location-Outcome	Latitude	Longitude	TOW %	OK/No/ Conditional
ADELAIDE (KENT TOWN)	-34.9211	138.6216	22	OK
ADELAIDE (WEST TERRACE / NGAYIRDAPIRA)	-34.9257	138.5832	20	OK
ADELAIDE AIRPORT	-34.9524	138.5196	21	OK
AIREYS INLET	-38.4583	144.0883	37	OK
ALBION PARK (SHELLHARBOUR AIRPORT)	-34.5639	150.7924	47	OK
ALBURY AIRPORT AWS	-36.069	146.9509	40	OK
ALICE SPRINGS AIRPORT	-23.7951	133.889	9	OK
ALVA BEACH	-19.4569	147.4833	43	OK
AMBERLEY AMO	-27.6297	152.7111	44	OK
APPLETHORPE	-28.6217	151.9533	50	OK
ARCHERFIELD AIRPORT	-27.5716	153.0071	35	OK
ARMIDALE AIRPORT AWS	-30.5273	151.6158	53	OK
AVALON AIRPORT	-38.0288	144.4783	43	OK
AYR DPI RESEARCH STN	-19.6169	147.3758	44	OK
BADGERYS CREEK AWS	-33.8969	150.7281	42	OK
BAIRNSDALE AIRPORT	-37.8818	147.5669	47	OK
BALLARAT AERODROME	-37.5127	143.7911	53	OK
BALLERA GAS FIELD	-27.4008	141.8114	6	OK
BALLINA AIRPORT AWS	-28.8353	153.5585	50	OK
BANKSTOWN AIRPORT AWS	-33.9176	150.9837	37	OK
BATHURST AIRPORT AWS	-33.4119	149.654	48	OK
BAYU-UNDAN (CUQ RIG)	-11.0681	126.6144	15	OK
BEAUDESERT DRUMLEY STREET	-27.9707	152.9898	48	OK
BEERBURRUM FOREST STATION	-26.9586	152.9619	52	OK
BEGA AWS	-36.6722	149.8191	49	OK
BELLAMBI AWS	-34.3692	150.9291	31	OK
BENDIGO AIRPORT	-36.7411	144.3274	31	OK
BIRDSVILLE AIRPORT	-25.8975	139.3472	5	OK
BLACKALL AIRPORT	-24.4303	145.4306	11	OK
BLACKWATER AIRPORT	-23.6015	148.8074	19	OK
BOMBALA AWS	-37.0015	149.2336	52	OK
BORROLOOLA AIRPORT	-16.0755	136.3041	33	OK
BOURKE AIRPORT AWS	-30.0362	145.9521	12	OK
BOWEN AIRPORT AWS	-20.0153	148.2138	43	OK

Location-Outcome	Latitude	Longitude	TOW %	OK/No/ Conditional
BRADSHAW	-14.9408	130.8091	18	OK
BRADSHAW - ANGALLARI VALLEY (DEFENCE)	-15.4397	130.5731	17	OK
BRADSHAW - KOOLENDONG VALLEY (DEFENCE)	-15.1853	130.1169	32	OK
BRAIDWOOD RACECOURSE AWS	-35.4253	149.7835	53	OK
BREAKWATER (GEELONG RACECOURSE)	-38.1737	144.3765	44	OK
BRISBANE	-27.4808	153.0389	31	OK
BRISBANE AERO	-27.3917	153.1292	39	OK
BROKEN HILL AIRPORT AWS	-32.0012	141.4694	11	OK
BULMAN	-13.6715	134.3415	34	OK
BUNDABERG AERO	-24.9069	152.323	44	OK
BUNGER HILLS	-66.251	100.6	6	OK
BURKETOWN AIRPORT	-17.7483	139.5356	26	OK
BUSHY PARK (BUSHY PARK ESTATES)	-42.7097	146.8983	41	OK
BUTLERS GORGE	-42.2753	146.2758	51	OK
BYRON BAY (CAPE BYRON AWS)	-28.6399	153.6358	41	OK
CABRAMURRA SMHEA AWS	-35.9371	148.3779	46	OK
CAIRNS AERO	-16.8736	145.7458	42	OK
CAIRNS RACECOURSE	-16.9463	145.7474	52	OK
CAMDEN AIRPORT AWS	-34.039	150.689	43	OK
CAMOOWEAL TOWNSHIP	-19.9225	138.1214	10	OK
CAMPANIA (KINCORA)	-42.6867	147.4258	30	OK
CAMPBELLTOWN (MOUNT ANNAN)	-34.0615	150.7735	47	OK
CANBERRA AIRPORT	-35.3106	149.197	39	OK
CANTERBURY RACECOURSE AWS	-33.9057	151.1134	40	OK
CANUNGRA (DEFENCE)	-28.0437	153.1871	59	OK
CAPE BORDA	-35.7549	136.5959	41	OK
CAPE BRUNY (CAPE BRUNY)	-43.4886	147.1444	30	OK
CAPE FLATTERY	-14.9672	145.3106	44	OK
CAPE GRIM BAPS (COMPARISON)	-40.6828	144.69	44	OK
CAPE JAFFA (THE LIMESTONE)	-36.9655	139.7164	39	OK
CAPE MORETON LIGHTHOUSE	-27.0314	153.4661	36	OK
CAPE NELSON LIGHTHOUSE	-38.4306	141.5437	40	OK
CAPE OTWAY LIGHTHOUSE	-38.8557	143.513	36	OK
CAPE SORELL	-42.1986	145.17	45	OK

9.1 Appendix B: Australian Time of Wetness Table (continued)

Location-Outcome	Latitude	Longitude	TOW %	OK/No/ Conditional
CAPE WESSEL	-11.0047	136.7586	45	OK
CAPE WILLOUGHBY	-35.8426	138.1327	36	OK
CARTERS BORE	-20.9358	139.2964	7	OK
CASEY	-66.2825	110.5231	2	OK
CASEY SKIWAY SOUTH	-66.2803	110.7615	1	OK
CASINO AIRPORT AWS	-28.8824	153.0618	51	OK
CASTERTON	-37.583	141.3339	56	OK
CEDUNA AMO	-32.1297	133.6976	34	OK
CENTRAL ARNHEM PLATEAU	-13.3275	133.0861	22	OK
CENTRE ISLAND	-15.7426	136.8192	29	OK
CENTURY MINE	-18.7569	138.7056	12	OK
CERBERUS	-38.3646	145.1785	35	OK
CESSNOCK AIRPORT AWS	-32.7914	151.3369	46	OK
CHARLEVILLE AERO	-26.4139	146.2558	12	OK
CHARLTON	-36.2846	143.3341	35	OK
CHRISTMAS ISLAND AERO	-10.4515	105.689	75	NO
CLARE HIGH SCHOOL	-33.8226	138.5933	34	OK
CLERMONT AIRPORT	-22.7756	147.6217	22	OK
CLEVE AERODROME	-33.7081	136.5026	33	OK
CLONCURRY AIRPORT	-20.6664	140.505	8	OK
COBAR AIRPORT AWS	-31.5388	145.7964	18	OK
COBAR MO	-31.484	145.8294	14	OK
COCOS ISLAND AIRPORT	-12.1892	96.8344	46	OK
COEN AIRPORT	-13.7606	143.1183	40	OK
COFFIN BAY (POINT AVOID)	-34.675	135.3362	47	OK
COFFS HARBOUR AIRPORT	-30.3189	153.1162	44	OK
COLAC (MOUNT GELLIBRAND)	-38.2333	143.7924	65	Conditional
COLDSTREAM	-37.7239	145.4092	52	OK
COMBIENBAR AWS	-37.3416	149.0227	38	OK
CONDOBOLIN AIRPORT AWS	-33.0682	147.2133	25	OK
COOBER PEDY AIRPORT	-29.0347	134.7222	8	OK
COOKTOWN AIRPORT	-15.4461	145.1861	47	OK
COOLANGATTA	-28.1681	153.5053	43	OK
COOMA AIRPORT AWS	-36.2939	148.9725	41	OK
COONABARABRAN AIRPORT AWS	-31.333	149.2699	27	OK
COONAMBLE AIRPORT AWS	-30.9776	148.3798	26	OK
COONAWARRA	-37.2906	140.8254	46	OK
COORANBONG (LAKE MACQUARIE AWS)	-33.0887	151.4636	55	OK

Location-Outcome	Latitude	Longitude	TOW %	OK/No/ Conditional
CORNER INLET (YANAKIE)	-38.8051	146.1936	60	OK
COWLEY BEACH (DEFENCE)	-17.6905	146.1126	58	OK
COWRA AIRPORT AWS	-33.8382	148.654	37	OK
CRESSY (BRUMBYS CREEK)	-41.7114	147.082	43	OK
CRESSY RESEARCH STATION	-41.7256	147.0794	46	OK
CULTANA (DEFENCE)	-32.6845	137.3689	21	OK
CUMMINS AERO	-34.2524	135.7135	45	OK
DALBY AIRPORT	-27.1605	151.2633	26	OK
DALY WATERS AIRSTRIP	-16.2637	133.3782	16	OK
DARTMOOR	-37.9222	141.2614	57	OK
DAVIS	-68.5744	77.9672	1	OK
DAVIS (WHOOP WHOOP)	-68.4723	78.8735	0	OK
DELAMERE WEAPONS RANGE	-15.7441	131.9181	15	OK
DENILIQUIN AIRPORT AWS	-35.5575	144.9458	27	OK
DENNES POINT	-43.0639	147.3567	25	OK
DEVONPORT AIRPORT	-41.1701	146.4289	47	OK
DOUBLE ISLAND POINT LIGHTHOUSE	-25.9319	153.1906	36	OK
DOUGLAS RIVER RESEARCH FARM	-13.8345	131.1872	40	OK
DUBBO AIRPORT AWS	-32.2206	148.5753	31	OK
DUNALLEY (STROUD POINT)	-42.9017	147.7894	23	OK
EAST POINT	-12.4108	130.8273	44	OK
EAST SALE	-38.1156	147.1323	49	OK
EAST SALE AIRPORT	-38.1016	147.1398	50	OK
EDENHOPE AIRPORT	-37.0222	141.2657	42	OK
EDINBURGH RAAF	-34.7111	138.6223	21	OK
EDITHBURGH	-35.1121	137.7395	39	OK
EILDON FIRE TOWER	-37.2091	145.8423	39	OK
EMERALD AIRPORT	-23.5694	148.1756	22	OK
ERNABELLA (PUKATJA)	-26.2635	132.1771	6	OK
ESSENDON AIRPORT	-37.7276	144.9066	39	OK
EUABALONG (MOUNT HOPE AWS)	-32.8261	145.8801	28	OK
EVANS HEAD RAAF BOMBING RANGE AWS	-29.183	153.3964	42	OK
FALLS CREEK	-36.8708	147.2755	42	OK
FERNY CREEK	-37.8748	145.3497	52	OK
FINGAL (FLEMING ST)	-41.643	147.98	40	OK
FINGAL (LEGGE STREET)	-41.6428	147.9664	31	OK
FLINDERS ISLAND AIRPORT	-40.0911	148.0024	47	OK
FORBES AIRPORT AWS	-33.3627	147.9205	35	OK

Location-Outcome	Latitude	Longitude	TOW %	OK/No/ Conditional
FOWLERS GAP AWS	-31.0863	141.7008	10	OK
FRANKSTON (BALLAM PARK)	-38.1517	145.1615	40	OK
FREDERICK REEF	-20.9375	154.4019	75	NO
FRIENDLY BEACHES	-41.9953	148.2794	33	OK
GABO ISLAND LIGHTHOUSE	-37.5679	149.9158	44	OK
GAYNDAH AIRPORT	-25.6167	151.6156	32	OK
GELANTIPY	-37.22	148.2626	52	OK
GEORGETOWN AIRPORT	-18.3039	143.5306	18	OK
GIRILAMBONE (OKEH) AWS	-31.0824	146.9296	21	OK
GLADSTONE AIRPORT	-23.8697	151.2214	23	OK
GLADSTONE RADAR	-23.8553	151.2628	35	OK
GLEN INNES AIRPORT AWS	-29.678	151.694	50	OK
GOLD COAST SEAWAY	-27.939	153.4283	37	OK
GOONDIWINDI AIRPORT	-28.5226	150.3223	30	OK
GOSFORD AWS	-33.4351	151.3614	49	OK
GOULBURN AIRPORT AWS	-34.8085	149.7311	52	OK
GOVE AIRPORT	-12.2684	136.8188	54	OK
GOVE AIRPORT MET OFFICE	-12.2741	136.8201	45	OK
GRAFTON AIRPORT AWS	-29.7583	153.0297	52	OK
GRAFTON RESEARCH STN	-29.6224	152.9605	51	OK
GRAMPIANS (MOUNT WILLIAM)	-37.295	142.6039	61	Conditional
GREEN CAPE AWS	-37.2622	150.0504	46	OK
GREENBANK (DEFENCE)	-27.6932	152.9935	52	OK
GRIFFITH AIRPORT AWS	-34.2489	146.0698	30	OK
GROOTE EYLANDT AIRPORT	-13.9746	136.463	49	OK
GROVE (RESEARCH STATION)	-42.9844	147.0756	41	OK
GUNNEDAH AIRPORT AWS	-30.9537	150.2494	26	OK
GUNNEDAH RESOURCE CENTRE	-31.0261	150.2687	17	OK
GYMPIE	-26.1831	152.6414	56	OK
HAMILTON AIRPORT	-37.6486	142.0636	51	OK
HAMILTON ISLAND AIRPORT	-20.3658	148.9536	45	OK
HARTZ MOUNTAIN (KEOGHS PIMPLE)	-43.2006	146.7683	73	NO
HAY AIRPORT AWS	-34.5412	144.8315	30	OK
HERVEY BAY AIRPORT	-25.322	152.8817	46	OK
HIGH RANGE AWS (WANGANDERRY)	-34.3335	150.267	53	OK
HINDMARSH ISLAND AWS	-35.5194	138.8177	30	OK
HOBART (ELLERSLIE ROAD)	-42.8897	147.3278	21	OK
HOBART AIRPORT	-42.8333	147.5119	33	OK

Location-Outcome	Latitude	Longitude	TOW %	OK/No/ Conditional
HOBART AIRPORT WEST	-42.8339	147.5033	24	OK
HOGAN ISLAND	-39.2225	146.9841	63	Conditional
HOLSWORTHY AERODROME AWS	-33.9925	150.9489	40	OK
HOLSWORTHY DEFENCE AWS	-34.081	150.9009	44	OK
HOPETOUN AIRPORT	-35.7151	142.3569	32	OK
HORN ISLAND	-10.5844	142.29	43	OK
HORSHAM AERODROME	-36.6699	142.1733	40	OK
HORSLEY PARK EQUESTRIAN CENTRE AWS	-33.851	150.8567	36	OK
HUGHENDEN AIRPORT	-20.8192	144.2333	10	OK
HUNTERS HILL	-36.2137	147.5395	39	OK
INGHAM AERO	-18.6651	146.1467	86	NO
INNISFAIL AERODROME	-17.5581	146.0119	69	Conditional
INVERELL RESEARCH CENTRE	-29.7752	151.082	28	OK
IVANHOE AERODROME AWS	-32.8833	144.3092	19	OK
JERVIS BAY (POINT PERPENDICULAR AWS)	-35.0936	150.8049	38	OK
JERVIS BAY NSW (JERVIS BAY AIRFIELD AWS)	-35.1439	150.6973	47	OK
JERVOIS	-22.9494	136.1442	7	OK
JULIA CREEK AIRPORT	-20.6672	141.7214	11	OK
KADINA AWS	-33.9703	137.6628	33	OK
KANAGULK	-37.1169	141.8031	47	OK
KANGAROO FLATS (DEFENCE)	-12.7934	130.8542	41	OK
KAPOOKA (DEFENCE)	-35.1327	147.2512	34	OK
KEITH (MUNKORA)	-36.1058	140.3273	38	OK
KELLALAC (WARRACKNABEAL AIRPORT)	-36.3203	142.4161	39	OK
KEMPSEY AIRPORT AWS	-31.0711	152.7717	58	OK
KENNAOOK/CAPE GRIM	-40.6764	144.6922	55	OK
KHANCOBAN AWS	-36.2304	148.1405	51	OK
KIAMA (BOMBO HEADLAND)	-34.6532	150.8609	22	OK
KING ISLAND AIRPORT	-39.8804	143.8857	50	OK
KINGAROY AIRPORT	-26.5737	151.8398	46	OK
KINGSCOTE AERO	-35.7114	137.5231	44	OK
KNUCKEY LAGOON	-12.4422	130.9556	50	OK
KOWANYAMA AIRPORT	-15.4818	141.7483	34	OK
KUITPO FOREST RESERVE	-35.1712	138.6783	44	OK
KUNANYI (MOUNT WELLINGTON PINNACLE)	-42.895	147.2358	58	OK
KYABRAM	-36.335	145.0638	41	OK
LADY ELLIOT ISLAND	-24.1116	152.7161	29	OK
LAJAMANU AIRPORT	-18.3324	130.6361	8	OK

9.1 Appendix B: Australian Time of Wetness Table (continued)

Location-Outcome	Latitude	Longitude	TOW %	OK/No/ Conditional
LAKE JULIUS AWS	-20.1167	139.7256	9	OK
LAMEROO (AUSTIN PLAINS)	-35.3778	140.5378	36	OK
LARAPUNA (EDDYSTONE POINT)	-40.9928	148.3467	48	OK
LAUNCESTON (TI TREE BEND)	-41.4194	147.1219	44	OK
LAUNCESTON AIRPORT	-41.5476	147.2156	43	OK
LAVERTON RAAF	-37.8565	144.7565	35	OK
LEIGH CREEK AIRPORT	-30.5963	138.4219	7	OK
LIAWENEE	-41.8997	146.6694	55	OK
LISMORE AIRPORT AWS	-28.8305	153.2601	54	OK
LOCHINGTON	-23.9424	147.525	17	OK
LOCKHART RIVER AIRPORT	-12.785	143.3047	57	OK
LONGERENONG	-36.6722	142.2991	40	OK
LONGREACH AERO	-23.4397	144.2828	9	OK
LORD HOWE ISLAND AERO	-31.5421	159.0786	35	OK
LOW HEAD	-41.0547	146.7874	51	OK
LOW ISLES LIGHTHOUSE	-16.3842	145.5592	45	OK
LOW ROCKY POINT	-42.9831	145.5025	51	OK
LOXTON RESEARCH CENTRE	-34.439	140.5978	24	OK
LUCINDA POINT	-18.5203	146.3861	19	OK
LUNCHEON HILL (FORESTRY)	-41.1492	145.1517	72	NO
MAATSUYKER ISLAND LIGHTHOUSE	-43.6578	146.2711	46	OK
MACKAY AERO	-21.1706	149.1794	48	OK
MACKAY M.O	-21.1172	149.2169	44	OK
MACQUARIE ISLAND	-54.4994	158.9369	73	NO
MAITLAND AIRPORT AWS	-32.7023	151.4881	49	OK
MALLACOOTA	-37.5976	149.7289	40	OK
MANGALORE AIRPORT	-36.8886	145.1859	37	OK
MANGROVE MOUNTAIN AWS	-33.2894	151.2107	52	OK
MANINGRIDA AIRPORT	-12.0569	134.2339	39	OK
MAREEBA AIRPORT	-17.0704	145.4293	50	OK
MARIA ISLAND (POINT LESUEUR)	-42.6621	148.0179	31	OK
MARRANGAROO (DEFENCE)	-33.4346	150.135	49	OK
MARREE AERO	-29.6587	138.0684	6	OK
MARYBOROUGH	-25.5132	152.7152	52	OK
MAWSON	-67.6017	62.8753	1	OK
MCARTHUR RIVER MINE AIRPORT	-16.4423	136.076	23	OK
MELBOURNE (OLYMPIC PARK)	-37.8255	144.9816	28	OK
MELBOURNE AIRPORT	-37.6654	144.8322	35	OK

Location-Outcome	Latitude	Longitude	TOW %	OK/No/ Conditional
MERIMBULA AIRPORT AWS	-36.9077	149.8989	52	OK
MERRIWA (ROSCOMMON)	-32.1852	150.1737	45	OK
MIDDLE PERCY ISLAND	-21.6628	150.2711	46	OK
MILDURA AIRPORT	-34.2358	142.0867	19	OK
MILES CONSTANCE STREET	-26.6569	150.1819	30	OK
MILINGIMBI AIRPORT	-12.0932	134.8919	38	OK
MINLATON AERO	-34.748	137.5276	35	OK
MINNIPA PIRSA	-32.8427	135.1515	24	OK
MONTAGUE ISLAND LIGHTHOUSE	-36.2519	150.2275	37	OK
MOOMBA AIRPORT	-28.0997	140.1956	7	OK
MOORABBIN AIRPORT	-37.98	145.0962	36	OK
MORANBAH AIRPORT	-22.0644	148.0758	28	OK
MOREE AERO	-29.4898	149.8471	19	OK
MORNINGTON ISLAND AIRPORT	-16.662	139.1655	35	OK
MORTLAKE RACECOURSE	-38.0737	142.7744	61	Conditional
MORUYA AIRPORT AWS	-35.9004	150.1437	44	OK
MORWELL (LATROBE VALLEY AIRPORT)	-38.2094	146.4746	49	OK
MOSS VALE AWS	-34.5253	150.4217	55	OK
MOUNT BAW BAW	-37.8384	146.2747	49	OK
MOUNT BOYCE AWS	-33.6185	150.2741	52	OK
MOUNT BULLER	-37.145	146.4394	41	OK
MOUNT BUNDEY NORTH (DEFENCE)	-12.9141	131.8663	33	OK
MOUNT BUNDEY SOUTH (DEFENCE)	-13.0889	131.8478	32	OK
MOUNT CRAWFORD AWS	-34.7253	138.9278	42	OK
MOUNT GAMBIER AERO	-37.7473	140.7739	52	OK
MOUNT GININI AWS	-35.5293	148.7721	47	OK
MOUNT HOTHAM	-36.9772	147.1342	39	OK
MOUNT ISA AERO	-20.6778	139.4875	9	OK
MOUNT LOFTY	-34.9784	138.7088	51	OK
MOUNT MOORNAPA	-37.7481	147.1428	46	OK
MOUNT NOWA NOWA	-37.6924	148.0908	41	OK
MOUNT READ	-41.8444	145.5417	77	NO
MOUNT STUART (DEFENCE)	-19.4082	146.762	41	OK
MUDGEE AIRPORT AWS	-32.5628	149.6149	37	OK
MULURULU AWS	-33.3392	143.4	19	OK
MURGANELLA AIRSTRIP	-11.5485	132.9266	53	OK
MURRAY BRIDGE (PALLAMANA AERODROME)	-35.065	139.2273	39	OK
MURRURUNDI GAP AWS	-31.7416	150.7937	40	OK

Location-Outcome	Latitude	Longitude	TOW %	OK/No/ Conditional
NAMBOUR DAFF - HILLSIDE	-26.6442	152.9383	48	OK
NARACOORTE AERODROME	-36.9813	140.727	45	OK
NARRABRI AIRPORT AWS	-30.3154	149.8302	19	OK
NARRANDERA AIRPORT AWS	-34.705	146.514	30	OK
NERRIGA AWS	-35.1103	150.0826	54	OK
NEW MAY DOWNS	-20.59	139.3411	8	OK
NEWCASTLE NOBBYS SIGNAL STATION AWS	-32.9184	151.7985	41	OK
NGAYAWILI	-11.9971	135.5726	50	OK
NHILL AERODROME	-36.3093	141.6486	33	OK
NILMA NORTH (WARRAGUL)	-38.1321	145.9865	59	OK
NOARLUNGA	-35.1586	138.5057	26	OK
NOONA AWS	-31.7267	144.9313	13	OK
NOONAMAH AIRSTRIP	-12.6099	131.0474	43	OK
NORAH HEAD AWS	-33.2814	151.5766	49	OK
NORFOLK ISLAND AERO	-29.0389	167.9408	40	OK
NORMANTON AIRPORT	-17.6872	141.0733	22	OK
NORTH SHIELDS (PORT LINCOLN AWS)	-34.5993	135.8784	37	OK
NOWRA RAN AIR STATION AWS	-34.9469	150.5353	46	OK
NULLARBOR	-31.4492	130.8976	37	OK
NULLO MOUNTAIN AWS	-32.7244	150.229	53	OK
NURIOOTPA PIRSA	-34.4761	139.0056	36	OK
OAKEY AERO	-27.4034	151.7413	42	OK
OENPELLI AIRPORT	-12.3272	133.0069	36	OK
OMEO	-37.1017	147.6008	49	OK
OODNADATTA AIRPORT	-27.5553	135.4456	5	OK
OORALEA RACECOURSE (MACKAY TURF CLUB)	-21.17	149.1515	51	OK
ORANGE AIRPORT AWS	-33.3768	149.1263	49	OK
ORBOST	-37.6922	148.4667	45	OK
OUSE FIRE STATION	-42.4842	146.7106	35	OK
PADTHAWAY SOUTH	-36.6539	140.5212	40	OK
PALMERVILLE	-15.9999	144.0754	37	OK
PARAFIELD AIRPORT	-34.7977	138.6281	24	OK
PARAWA (SECOND VALLEY FOREST AWS)	-35.5695	138.2864	55	OK
PARKES AIRPORT AWS	-33.1281	148.2428	32	OK
PARNDANA CFS AWS	-35.7916	137.2496	49	OK
PATERSON (TOCAL AWS)	-32.6296	151.5919	44	OK
PENRITH LAKES AWS	-33.7195	150.6783	49	OK
PERISHER VALLEY AWS	-36.4069	148.4055	37	OK

Location-Outcome	Latitude	Longitude	TOW %	OK/No/ Conditional
POINT COOK RAAF	-37.9273	144.7566	34	OK
POINT FAWCETT	-11.7628	130.03	48	OK
PORT AUGUSTA AERO	-32.5073	137.7169	14	OK
PORT FAIRY AWS	-38.3906	142.2348	48	OK
PORT KEATS AIRPORT	-14.2494	129.5282	34	OK
PORT MACQUARIE AIRPORT AWS	-31.4343	152.8662	56	OK
PORT MACQUARIE AIRPORT AWS (COMPARISON)	-31.4335	152.8655	49	OK
PORT PIRIE AERODROME AWS	-33.2371	137.9971	23	OK
PORTLAND AIRPORT	-38.3148	141.4705	46	OK
POUND CREEK	-38.6297	145.8107	51	OK
PROSERPINE AIRPORT	-20.4925	148.555	56	OK
PUCKAPUNYAL LYON HILL (DEFENCE)	-36.9381	145.0539	36	OK
PUCKAPUNYAL WEST (DEFENCE)	-37.0177	144.8546	37	OK
PYRENEES (BEN NEVIS)	-37.2281	143.2005	53	OK
RABBIT FLAT	-20.1823	130.0148	9	OK
REDCLIFFE	-27.2169	153.0922	36	OK
REDESDALE	-37.0194	144.5203	41	OK
REDLAND (ALEXANDRA HILLS)	-27.5433	153.2394	42	OK
RENMARK AERO	-34.1983	140.6766	19	OK
RHYLL	-38.4612	145.3101	34	OK
RICHMOND AIRPORT	-20.7001	143.1137	10	OK
RICHMOND RAAF	-33.6004	150.7761	51	OK
ROBE AIRFIELD	-37.1776	139.8054	47	OK
ROCKHAMPTON AERO	-23.3753	150.4775	31	OK
ROLLESTON AIRPORT	-24.4617	148.6264	20	OK
ROMA AIRPORT	-26.5477	148.771	23	OK
ROSEWORTHY AWS	-34.5106	138.6763	34	OK
ROXBY DOWNS (OLYMPIC DAM AERODROME)	-30.4869	136.8739	8	OK
RUTHERGLEN RESEARCH	-36.1048	146.5094	42	OK
SAMUEL HILL AERO	-22.7433	150.6578	56	OK
SCHERGER RAAF	-12.6167	142.0869	47	OK
SCONE AIRPORT AWS	-32.0335	150.8264	43	OK
SCORESBY RESEARCH INSTITUTE	-37.871	145.2561	41	OK
SCOTTS PEAK DAM	-43.0425	146.2722	70	Conditional
SCOTTSDALE (WEST MINSTONE ROAD)	-41.1708	147.4883	50	OK
SHE OAKS	-37.9075	144.1303	44	OK
SHEFFIELD SCHOOL FARM	-41.389	146.3173	46	OK
SHEPPARTON AIRPORT	-36.4288	145.3949	39	OK

9.1 Appendix B: Australian Time of Wetness Table (continued)

Location-Outcome	Latitude	Longitude	TOW %	OK/No/ Conditional
SINGLETON DEFENCE AWS	-32.6976	151.1564	35	OK
SMITHTON AERODROME	-40.8347	145.0847	63	Conditional
SMITHVILLE AWS	-30.0692	141.0067	8	OK
SNOWTOWN (RAYVILLE PARK)	-33.7675	138.2182	32	OK
SOUTH JOHNSTONE EXP STN	-17.6053	145.9972	67	Conditional
ST GEORGE AIRPORT	-28.0478	148.5957	14	OK
ST HELENS AERODROME	-41.3381	148.2792	42	OK
ST LAWRENCE	-22.3472	149.5242	38	OK
STAWELL AERODROME	-37.072	142.7402	36	OK
STENHOUSE BAY	-35.2795	136.9392	25	OK
STRAHAN AERODROME	-42.155	145.2908	58	OK
STRATHALBYN RACECOURSE	-35.2836	138.8934	29	OK
SUNSHINE COAST AIRPORT	-26.599	153.0912	45	OK
SWAN HILL AERODROME	-35.3766	143.5416	27	OK
SWAN ISLAND	-40.7292	148.125	46	OK
SYDNEY (OBSERVATORY HILL)	-33.8607	151.205	19	OK
SYDNEY (OBSERVATORY HILL)	-33.8593	151.2048	42	OK
SYDNEY AIRPORT AMO	-33.9465	151.1731	30	OK
SYDNEY OLYMPIC PARK AWS (ARCHERY CENTRE)	-33.8338	151.0718	45	OK
TAMWORTH AIRPORT AWS	-31.0742	150.8362	34	OK
TARCOOLA AERO	-30.7051	134.5786	10	OK
TAREE AIRPORT AWS	-31.8895	152.512	53	OK
TASMAN ISLAND	-43.2397	148.0025	39	OK
TATURA INST SUSTAINABLE AG	-36.4379	145.2673	38	OK
TEMORA AIRPORT	-34.4277	147.5117	34	OK
TENNANT CREEK AIRPORT	-19.6423	134.1833	6	OK
TERREY HILLS AWS	-33.6908	151.2253	46	OK
TERRITORY GRAPE FARM	-22.4518	133.6377	7	OK
TEWANTIN RSL PARK	-26.3911	153.0403	48	OK
THANGOOL AIRPORT	-24.4935	150.5709	32	OK
THARGOMINDAH AIRPORT	-27.9867	143.815	7	OK
THE MONUMENT AIRPORT	-21.8125	139.9267	7	OK
THREDBO AWS	-36.4917	148.2859	37	OK
TIBOOBURRA AIRPORT	-29.4448	142.0567	9	OK
TIN CAN BAY (DEFENCE)	-25.9351	152.9647	48	OK
TINDAL RAAF	-14.5229	132.3826	19	OK
TOOWOOMBA AIRPORT	-27.5425	151.9134	46	OK
TOWNSVILLE AERO	-19.2483	146.7661	37	OK

Location-Outcome	Latitude	Longitude	TOW %	OK/No/ Conditional
TOWNSVILLE- AIR WEAPONS RANGE (DEFENCE)	-19.3048	146.2438	44	OK
TOWNSVILLE- FANNING RIVER (DEFENCE)	-19.783	146.5	41	OK
TRANGIE RESEARCH STATION AWS	-31.9861	147.9489	25	OK
TREPELL AIRPORT	-21.84	140.8925	8	OK
TUGGERANONG (ISABELLA PLAINS) AWS	-35.4184	149.0937	32	OK
TUNNACK FIRE STATION	-42.4543	147.4612	49	OK
ULLADULLA AWS	-35.3635	150.4828	33	OK
UNIVERSITY OF QUEENSLAND GATTON	-27.5436	152.3375	36	OK
URANDANGI AERODROME	-21.5979	138.3665	6	OK
VARANUS ISLAND	-20.655	115.5769	24	OK
VICTORIA RIVER DOWNS	-16.403	131.0145	12	OK
VIEWBANK	-37.7408	145.0972	45	OK
WAGGA WAGGA AMO	-35.1583	147.4575	35	OK
WALGETT (BREWON AWS)	-30.2411	147.5327	20	OK
WALGETT AIRPORT AWS	-30.0372	148.1223	20	OK
WALLAN (KILMORE GAP)	-37.3807	144.9654	56	OK
WALPEUP RESEARCH	-35.1201	142.004	25	OK
WALUNGURRU AIRPORT	-23.2656	129.3844	4	OK
WANAARING (BORRONA DOWNS AWS)	-29.7614	143.1135	10	OK
WANAARING (DELTA AWS)	-30.1006	145.3338	11	OK
WANGARATTA AERO	-36.4205	146.3056	41	OK
WARRA	-43.0609	146.704	56	OK
WARRNAMBOOL AIRPORT NDB	-38.2869	142.4524	54	OK
WARRUWI AIRPORT	-11.65	133.3797	33	OK
WARWICK	-28.2061	152.1003	39	OK
WEIPA AERO	-12.6778	141.9208	45	OK
WEST WYALONG AIRPORT AWS	-33.9382	147.1962	28	OK
WESTMERE	-37.7067	142.9378	55	OK
WHITE CLIFFS AWS	-30.8522	143.0743	9	OK
WHYALLA AERO	-33.0539	137.5206	24	OK

9.2 Appendix C: Dealing with Graffiti

WARNING:
This chemical is formulated for industrial use

Contact with skin or clothing or other improper handling or use of this product may result in bodily harm or other damage. Before using or mixing the contents with other substances, all labels applied to container, the applicable Technical Data Sheet and Material Safety Data Sheet should be read and specific instructions and precautions followed to assure correct use and personal safety.

Graffiti Red

Paint Stripper

Introduction

Graffiti Red is a mohogany coloured paint stripper. Graffiti Red is designed for removal of spray can paint, 'ironlak', buff ink, molotow and permanent markers from barebrick, masonry, concrete, stone and other non painted surfaces.

Application

1. Put on safety glasses, safety gloves and relevant PPE.
2. Pour some Red Graffiti Remover into a bucket and generously apply the remover to the paint with a nylon bristle brush or broom. Do not scrimp on the amount of product. Apply liberally.
3. Repeat this procedure two times in three-minute intervals for the majority of spray cans. There is no need to scrub hard, but agitation is certainly helpful. Extra applications may be necessary on porous surfaces or on old paint.
4. Keep the paint wet with the product, if it dries out, the paint remover will not work effectively... the longer, the wetter the better.
5. Rinse the dissolved paint off with pressure nozzle, trigger hose or pressure washer on low.

Important

Always read the safety instructions before using any cleaning product and observe all safe working requirements eg: safety gloves, safety glasses etc.

Wear Long Sleeves Do not apply chemical above shoulder height.

Do not blast too close in order to remove the paint as it may cause damage to the surface and will possibly only remove a small amount of the paint.

Graffiti Red Technical Data Sheet courtesy of BMG Chemicals Pty Ltd.

9.3 Appendix D: Prevention of Bird Nesting

Pinwire systems for the prevention of bird nesting

An almost invisible tensioned bird wire system to prevent birds perching without damaging the aesthetics or construction.

Eco's Bird Wire is one of the most discrete and widely used anti-perching systems available in the UK. It is effective to prevent both feral pigeons and seagulls from perching on the ledges and parapets.

Eco's Bird Wire System consists of a fine, nylon-coated, spring-tensioned stainless-steel wire supported by stainless steel posts, which are fixed into stonework using nylon anchor rivets.

The posts are available in various sizes. The posts, springs, wires and split pins are made of corrosion resistant 316-grade stainless steel.

Eco's Bird Wire is a very versatile Bird Control System and a variety of special components and brackets are available to enable it to be fitted to almost any surface, making it a Bird Deterrent System suitable for many applications.

Courtesy of Clearwell Pest Control Services

9.4 Appendix E: Part Turn Tightening Method Weathering Steel to AS/NZS 5131

1. Preloading Suitability

Preloading suitability is the testing of structural bolting assemblies to demonstrate that all components of the assembly (bolt, nut, washer) are able to be tightened using the methods prescribed to achieve the minimum bolt tension specified.

Two systems of demonstrating preloading suitability currently exist.

- Preloading suitability tested by supplier who first puts product to market
- Pre installation testing performed by installers on job site

System 1 is preferred in Australia and New Zealand as it is specified by AS/NZS 1252.1. For weathering steel structural assemblies to ASTM F31252/M (ASTM A325/A490) testing to ASTM F3125 Annex A2 – Rotational Capacity Testing shall be specified. For weathering steel structural assemblies to EN 14399 testing to EN 14399-2 shall be specified.

Weathering steel bolt assemblies imported from USA are likely to come as unmatched assemblies (no assembly testing performed). Onsite pre installation verification (System 2) shall be completed in accordance with Addendum A.

Pre-installation verification testing shall be performed in compliance with all of the following:

1) At the site of installation;
2) Prior to the placement of bolting assemblies in the work;
3) On a sample of not fewer than three complete bolting assemblies of each combination of diameter, length, grade, and lot to be used in the work;
4) Using bolting assemblies that are representative of the condition of those that will be pretensioned in the work;
5) In accordance with the test procedure in Addendum A of this document.

Before installation of structural bolt assemblies is undertaken, test certificates shall be inspected to ensure that preloading suitability testing has been conducted on the bolt assemblies to be installed.

2. Snug Tightening

The intention of snug tightening is to achieve **firm contact**, which is defined as: the condition that exists between plies in a bolted connection where the plies are solidly seated against each other, but not necessarily in continuous contact, over the effective firm contact area. Where, the Effective Firm Contact Area is defined as the area within one bolt diameter of any holes but not less than 25 mm from the edge of any hole and **all areas within the bolt group.**

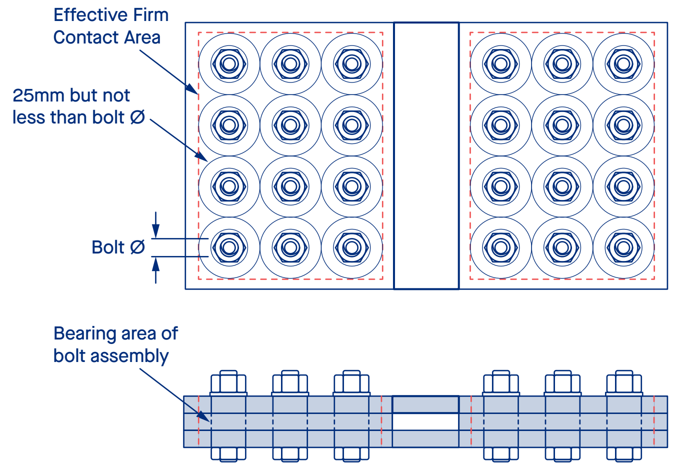
In weathering steel connections, gaps should be avoided within the bolt group.

Snug-tightening of the bolts in the connection should proceed systematically from the most rigid part of the connection towards the free edges. More than one cycle of snug-tightening may be required.

Snug tight is the tightness in the bolts in a bolted connection attained by a few impacts of an impact wrench or by the full effort of a person using a standard podger spanner to bring the plies into firm contact.

Steelwork between the bolt head and nut should be in contact. Gaps between the bolt head and nut are likely to result in bolts not achieving the minimum tension requirements when performing the part turn of nut tightening method.

Where firm contact cannot be achieved the gaps within the steelwork should be packed using steel packing plates or shims.



Tightening Procedure AS/NZS 5131
Courtesy of Hobson Engineering Co. Pty. Ltd.

9.4 Appendix E: Part Turn Tightening Method Weathering Steel to AS/NZS 5131 (continued)

3. Tightening Methods

Only the Part Turn of Nut and Direct Tension Indicator (DTI) Method are included in AS/NZS 5131. Weathering steel design guides do not recommend using the DTI method due to risk of crevice corrosion.

The Part-Turn of Nut method is a strain-controlled method. Strain-control that reaches the inelastic region of bolt behaviour is inherently more reliable than a method that is completely dependent upon torque control. Proper implementation is dependent upon ensuring that the joint is properly compacted prior to application of the required partial turn and that the bolt head (or nut) remains stationary when the nut (or bolt head) is being turned.

4. Part Turn of Nut Method

Step 1: Match Marking
After completing snug-tightening, location marks shall be established to mark the relative positions of the bolt and the nut. Best practice method of marking is to mark the bolt end radially out from the centre of the bolt end, across the nut at a corner and on the steelwork. This demonstrates that only the nut has rotated during the tightening operation.

Location marks should be permanent enough for an inspector to observe at a time after tensioning has been complete. This is to prevent disputes later as there is no way to reliably measure bolt tension after tensioning has been completed.

Step 2: Final Rotation
Bolts shall be tensioned by rotating the nut by the amount given in AS/NZS 5131 Table 8.5.6. During final tensioning, the component not turned by the wrench shall not rotate.

Step 1: Match Marking

Step 2: Final Rotation

Table 8.5.6: Nut rotation from the snug-tight condition

Bolt length (underside of head to end of bolt)	Disposition of outer face of bolted parts (see Notes 1, 2, 3 and 4)		
	Both faces normal to bolt axis	One face normal to bolt axis and other sloped	Both faces sloped
Up to and including 4 diameters	1/3 turn	1/2 turn	2/3 turn
Over 4 diameters but not exceeding 8 diameters	1/2 turn	2/3 turn	5/6 turn
Over 8 diameters but not exceeding 12 diameters (see Note 5)	2/3 turn	5/6 turn	1 turn

Notes:
1. Tolerance on rotation: for 1/2 turn or less, 1/12 of a turn (30°) over and nil under tolerance; for 2/3 turn or more, 1/8 of a turn (45°) over and nil under tolerance.
2. The bolt tension achieved with the amount of nut rotation specified in Table 8.5.6 will be at least equal to the minimum bolt tension specified in Table 8.5.5.
3. Nut rotation is the rotation relative to the bolt, regardless of the component turned
4. Nut rotations specified are only applicable to connections in which all material within the grip of the bolt is steel.
5. No research has been performed to establish the turn-of-nut procedure for bolt lengths exceeding 12 diameters. Therefore, the required rotation should be determined by actual test in a suitable tension measuring device which simulates conditions of solidly fitted steel. The 'assembly test' specified in AS/NZS 1252.1 is a suitable test.

Addendum A

Pre-installation Testing for Turn-of-Nut Method

Step 1: Snug-Tightening
The bolting assembly shall be installed to the snug-tight condition in the bolt tension measurement device using the tools, bolting components, assembly configuration, and installation methods to be used in the work.

Step 2: Matchmarking
If matchmarking is to be used in the work, the bolting assembly shall be matchmarked.

Step 3: Pre-tensioning
The rotation specified in AS/NZS 5131 Table 8.5.6 shall be applied to the bolting assembly.

Step 4: Final Verification
If the actual pretension developed in the bolting assembly is less than that specified, the cause(s) shall be determined and resolved before the bolting assemblies are used in the work. Cleaning, lubrication, and retesting of these bolting assemblies is permitted provided that all assemblies are treated in the same manner.

For further information,
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