Manual

Structures Inspection Manual
Part 2: Deterioration Mechanisms

September 2016

Queensland Government

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1 Introduction

The intent of any structure inspection is to identify and record the presence of defects. While diagnosis of the causes of defects is not a requirement for a Level `1 or Level 2 inspection, it is of great value for the inspector to have an appreciation of structural behaviour and of the defects that might occur.

Such an appreciation will guide and alert the inspector to particular signs enabling attention to be focussed where it is most needed. This ensures that, when a defect is observed, the necessary data is collected on site so that a correct diagnosis can be made, especially when defects occur due to a combination of causes.

Identification of structural defects and their causes require considerable care as structural distress within an element may often have consequential effects on other elements and it may not be immediately apparent which element has caused the failure. For example:

- Failure in the bridge foundations, due to settlement, sliding or rotation, is often manifested as cracking, differential movement or other defect in the substructure. Such movements may be displayed as abnormal clearances between the abutment ballast wall and the end of the deck, or as out-of-range movements at the expansion joints or bearings.
- Settlement of embankments may affect the substructure, appearing as depressions in the road surface adjacent to the structure or as discontinuities in the kerb line.
- Seized or locked bearings can transfer unexpected forces into the bearing shelf resulting in spalling to the front face of the headstock or bearing shelf.

This document is intended as a guide to some of the more common material and structure related defects likely to be encountered on the network.

2 Material defects

2.1 General

This section describes the defects that are normally found in concrete, steel, timber, masonry and coatings. Each defect is briefly described and the causes producing it are identified.

2.2 Concrete

Based on concrete defects described in Ontario Ministry of Transportation, *Ontario Structure Inspection Manual* and adjusted for Queensland conditions.

Concrete is used in structures as plain concrete, such as tremie and mass concrete; or it is combined with conventional steel reinforcement as reinforced concrete, or with prestressing steel reinforcement as prestressed concrete.

Defects in concrete can often be related to the lack of durability of the concrete, resulting from the composition of the concrete, poor placement practices, poor quality control or the aggressive environment in which it is placed.

The following defects which have occurred in the Queensland road infrastructure are described. They have been listed in order of occurrence from most common to least as found in our concrete road bridges to date:

i. Corrosion of reinforcement

ii. Carbonation[1]

iii. Alkali-aggregate Reaction(AAR)[2]
iv. Cracking
v. Spalling
vi. Surface defects
vii. Delamination
viii. Scaling
ix. Disintegration
x. Chloride ingress[1]
xi. Water wash.

[1] These phenomena cause the conditions for corrosion (and ultimately cracking, spalling and delamination) to occur.

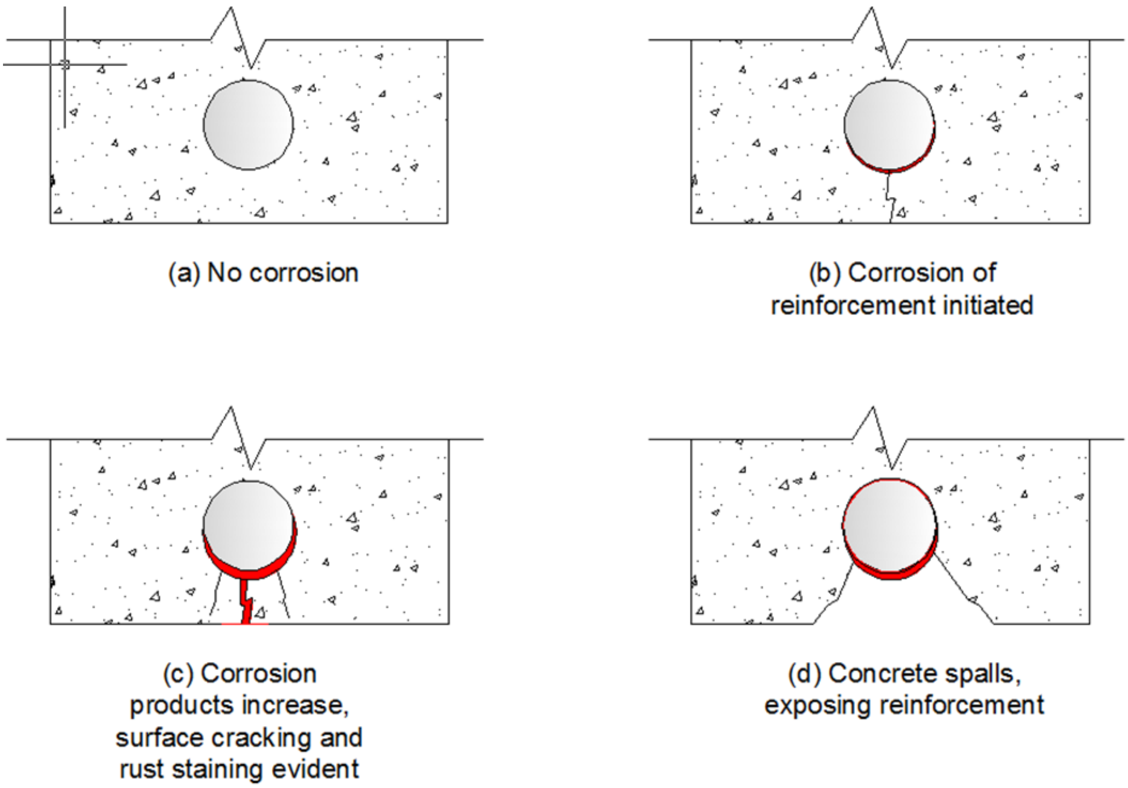
[2] This phenomena results in cracking of concrete and also promotes other deterioration mechanisms.

2.2.1 Corrosion of reinforcement

Corrosion is a consequence of the deterioration of reinforcement by electrolysis.

Reinforcement is usually protected by a passive film which forms due to the alkaline environment of concrete. Corrosion will not normally occur in most concrete elements unless the passive layer breaks down. This will typically only occur if the alkalinity of the concrete is reduced (through carbonation) or where sufficiently high concentrations of chloride in the concrete causes local break down of the passive layer.

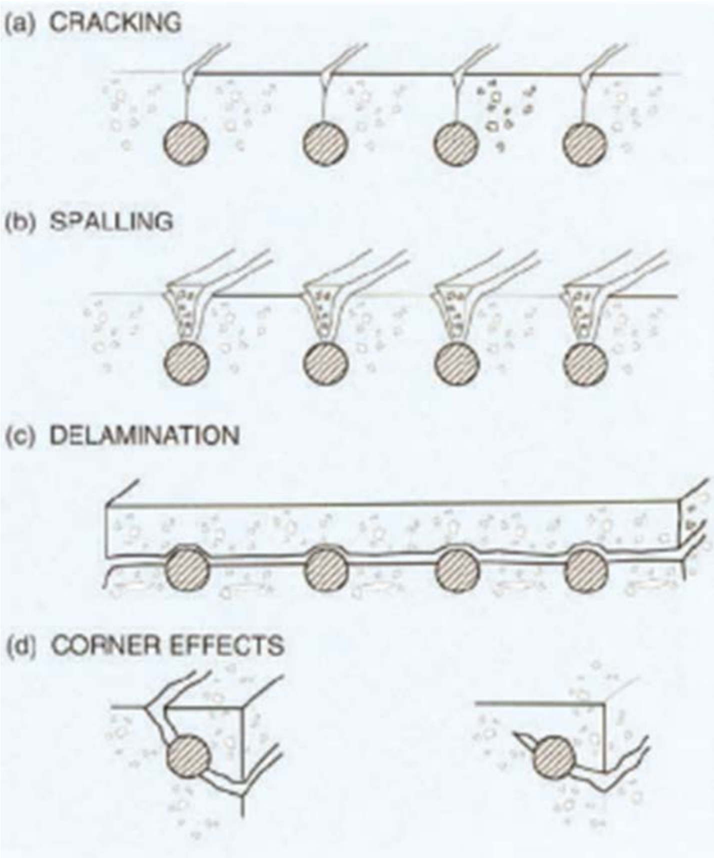
Figure 2.2.1a – Stages of reinforcement corrosion



In the initial stages, corrosion may appear as rust staining on the concrete surface. In the advanced stages, the cover concrete above the reinforcement cracks, delaminates and spalls off exposing heavily corroded reinforcement. This process is illustrated in Figure 2.2.1a.

Figure 2.2.1b illustrates types of damage sustained by concrete structures when embedded steel corrodes.

Figure 2.2.1b – Concrete defects resulting from reinforcement corrosion



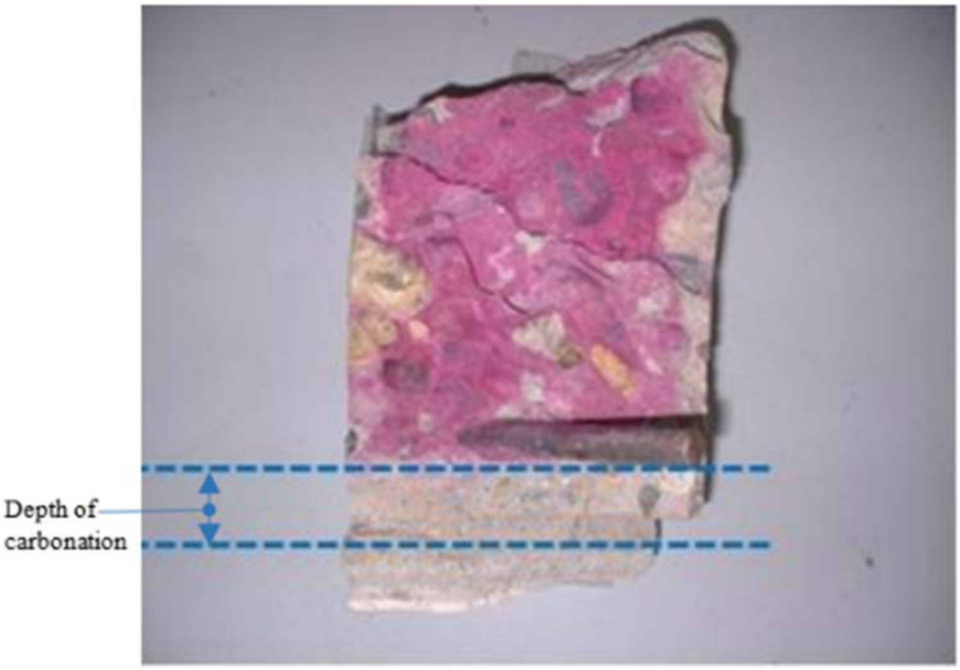
In Queensland, the most common example of damage resulting from reinforcement corrosion is found in the square section reinforced concrete piles which were used extensively until the introduction of prestressed octagonal piles. Cracking typically follows the line of the corner reinforcement where two concrete faces are exposed to the environment (increasing the rate of carbonation and/or chloride ingress) and the density of the concrete is compromised by limited access for compaction. The severity of the cracking increases until the cover concrete delaminates and ultimately spalls off exposing the corroded reinforcement. In addition horizontal cracking caused by driving stresses is often found in this type of pile.

2.2.1.1 Carbonation

Carbonation is the process by which carbon dioxide in the atmosphere dissolves in moisture within the concrete pores and reacts with calcium hydroxide present in the cement paste. This forms a neutral calcium carbonate which, over a long period of time gradually lowers the alkalinity of the concrete cover to the steel reinforcement, breaking down the protective passive film around the embedded steel.

The depth of carbonation from the exterior surface can be estimated by using a pH indicator, e.g. phenolphthalein dissolved in water. When the indicator is applied to a freshly exposed concrete surface the carbonated zone remains clear while the uncarbonated area turns pink (Figure 2.2.1.1).

Figure 2.2.1.1 – Example use of phenolphthalein indicator



As stated above, carbonation only provides an environment for corrosion to occur (i.e. by breaking down the passive protective layer) and does not, in itself, constitute a defect.

2.2.1.2 Chloride ingress

Chloride ions can initiate corrosion in embedded steel even at high alkanities (i.e. before the process of carbonation has reduced alkaline levels to the point where the passive protective layer around embedded steel has broken down).

The primary source for these ions to enter the concrete in Queensland is through contact with salt spray and/or salt-laden water. In coastal areas and river estuaries tidal flows can bring salt-laden water inland and/or strong winds may blow salt spray several kilometres inland.

Other sources include the introduction of chloride during concrete production through contaminated water or admixtures.

2.2.2 Alkali-Aggregate Reaction (AAR)

Two types of alkali-aggregate reaction are currently recognised depending on the nature of the reactive mineral. These are:

- Alkali-Silica Reaction (ASR) – involves various types of reactive silica.
- Alkali-Carbonate Reaction (ACR) – involves certain types of dolomitic rocks.

The reactive mineral present in aggregates react adversely with alkalis in the cement paste to produce hydrophilic gel within the pores of the concrete matrix. When the gel is exposed to moisture it expands, resulting in expansive forces being exerted on the concrete with associated deterioration in the form of cracking and spalling. The cracking occurs through the entire mass of the concrete and can take the form of extensive map cracking and/or cracking aligned with the major stress direction or reinforcement. White or colourless exudations around some of the cracks are also an indicator that AAR may be occurring. The appearance of concrete components affected by AAR is shown in Figure 2.2.2a – Figure 2.2.2d.

Alkali-aggregate reactions are generally slow by nature, and the results may not be apparent for 5 to 10 years.

It's important to note that water is required for the expansive process and, if this can be controlled, the effects of AAR can usually be managed.

Once AAR cracking presents, clearly it renders the concrete more vulnerable to deterioration through other processes (e.g. corrosion) by compromising the cover concrete.

ASR cracking most commonly occurs in prestressed deck units in Queensland (Figure 2.2.2d). Vertical cracking has also been detected in some prestressed octagonal piles which is the result of Alkali-Silica Reaction (ASR). The risk of this type of cracking has been minimised in new construction by the use of an appropriate mass of fly ash in the approved mix designs.

Figure 2.2.2a – Example ASR crack pattern in pier column

Figure 2.2.2b – Example ASR crack pattern in abutment wall

Figure 2.2.2c – Example ASR crack pattern in pier headstock

Figure 2.2.2d – Example ASR crack pattern in prestressed deck unit soffit

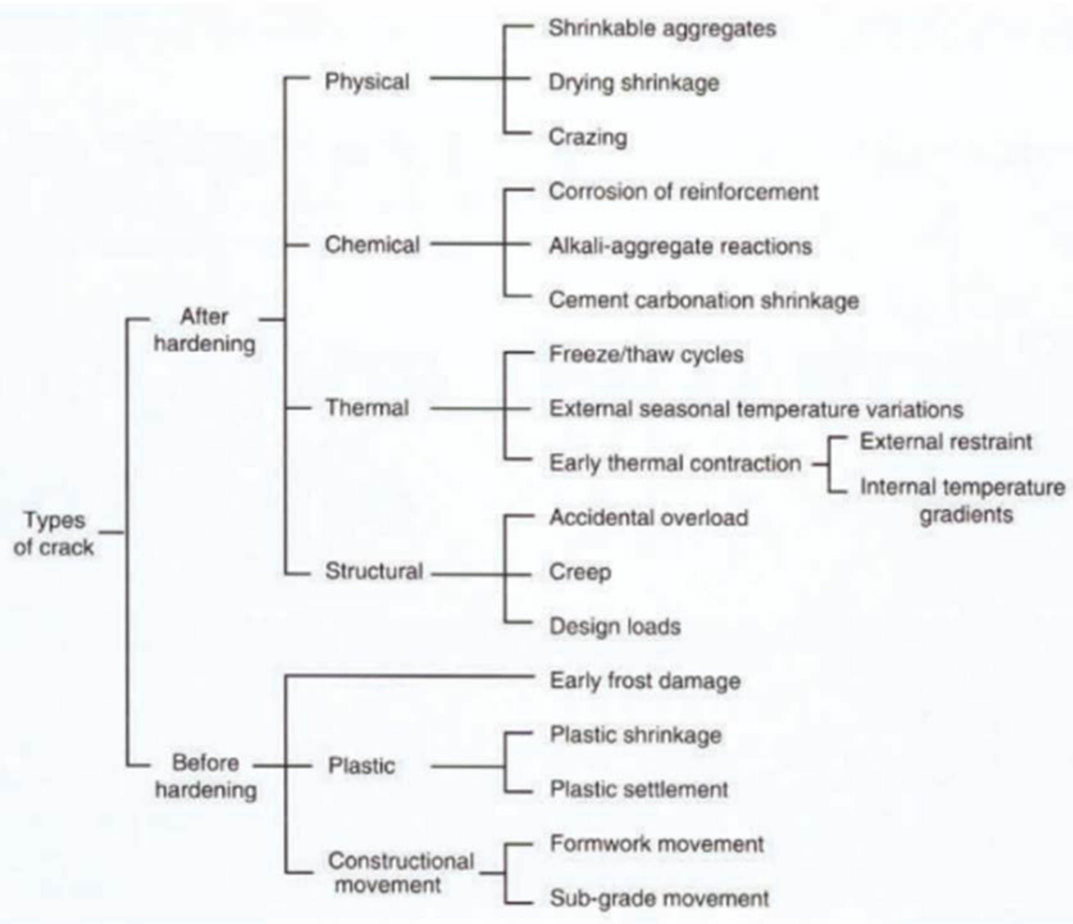
2.2.3 Cracking

A crack is a linear fracture in concrete which extends partly or completely through the member. Cracks in concrete occur due to tensile stresses introduced in the concrete as a result of volumetric changes or applied loads.

Tensile stresses are initially carried by the concrete and reinforcement until the level of the tensile stress exceeds the tensile capacity of the concrete. After this point the concrete cracks and the tensile force is transferred completely to the steel reinforcement. In reinforced and prestressed concrete, crack widths and their distribution are controlled by the reinforcing steel, whereas in plain concrete there is no such control.

The build-up of tensile stresses and, therefore, cracks in the concrete may be due to any number of causes and occur at different stages of the concrete development. Figure 2.2.3 summarises the possible crack types and the stages at which they may develop (plastic state and after hardening). Figure 2.2.3.1a illustrates typical crack patterns and locations.

Figure 2.2.3 – Types of cracking in concrete

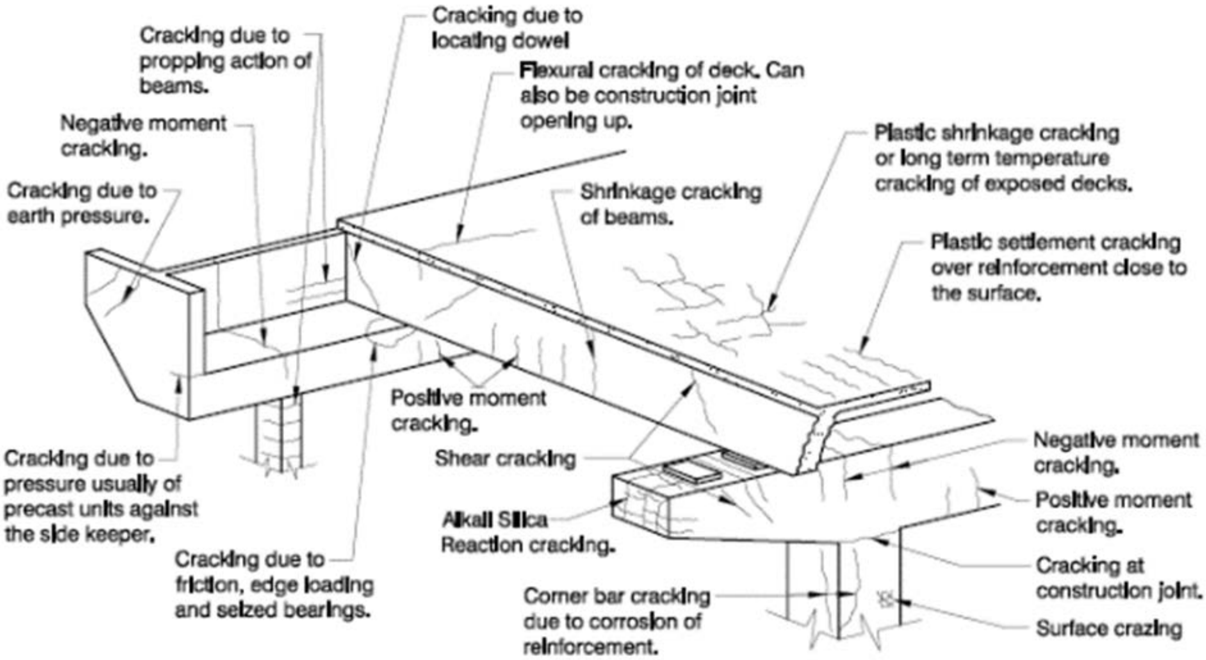


2.2.3.1 Plastic cracking

Plastic cracking of concrete occurs either as plastic settlement cracking or as plastic shrinkage cracking. Plastic settlement cracking typically occurs in columns, beams or walls, whilst plastic shrinkage cracks occur most commonly in freshly placed flat exposed slabs. Settlement induced cracking can also occur in non-vibrated slabs.

Both plastic and settlement cracking are associated with bleeding of fresh concrete, i.e. water rising to the top of the concrete shortly after compaction. It is caused by the gravitational pull on heavy solid particles in fresh concrete and the displacement of the lighter water upwards which causes bleeding.

Figure 2.2.3.1a – Classification of cracks in concrete structures



Plastic shrinkage cracks (refer Figure 2.2.3.1b) are typically short and random, forming a map pattern (crazing), a series of parallel lines or over reinforcement. These cracks, together with those resulting from thermal effects (in the plastic state) are typically shallow and are dormant. However, if a slab is significantly affected by plastic shrinkage cracking the cracks may continue through the depth of the slab.

Plastic settlement cracks (refer Figure 2.2.3.1c) occur opposite rigidly supported reinforcement or other embedded items. They may also occur at pronounced changes in section depth. They present as cracks following the direction of reinforcement on the tops of deep beams and slabs or stirrups in columns. The cracks can be wide at the surface but are rarely deep enough to affect the structural integrity.

Figure 2.2.3.1b – Plastic shrinkage cracking in bridge deck

Figure 2.2.3.1c – Plastic settlement crack in slab

2.2.3.2 Cracking of hardened concrete

Common causes of cracking in hardened concrete are:

- Drying shrinkage – essentially the contraction that occurs when fresh concrete rapidly dries. Concrete tends to shrink whenever its surfaces are exposed to air of relatively low humidity. Drying shrinkage cracks can present as longitudinal cracks (in the case of thin slabs and walls) or as a network of very fine, closely spaced random cracks (surface crazing).
- Steel corrosion – discussed in paragraph 2.2.1.
- Alkali-aggregate reactions – discussed in paragraph 2.2.2.
- Externally applied loads – these generate a system of internal compressive and tensile stresses, in the members and components of the structure, as required to maintain static equilibrium. For example, prestressing generates bursting effects at anchorage zones which will cause tensile cracks if the member is inadequately reinforced as shown in Figure 2.2.3.2a. Cracks resulting from externally applied loads initially appear as hairline cracks and are harmless. However as the reinforcement is further stressed the initial cracks open up and progressively spread into wider cracks (refer to Figure 2.2.3.2b and Figure 2.2.3.2c). Of particular concern is the development of shear cracks in structural members adjacent to supports which may be indicative of incipient brittle failure as shown in Figure 2.2.3.2d.
- External restraint forces - are generated if the free movement of the concrete in response to the effects of temperature, creep and shrinkage is prevented from occurring due to restraint at the member supports. The restraint may consist of friction at the bearings, bonding to already hardened concrete, or by attachment to other components of the structure. Cracks resulting from the actions of external restraint forces develop in a similar manner as those due to externally applied loads.

Figure 2.2.3.2a – Bursting cracks in under-reinforced anchorage zone of post-tensioned girder

Figure 2.2.3.2b – Typical flexural cracks at mid-span of girder

- Differential movement/settlement - differential movements or settlements result in the redistribution of external reactions and internal forces in the structure. This may in turn result in the introduction of additional tensile stresses and, therefore, cracking in the concrete components of the structure. Movement cracks may be of any orientation and width, ranging from fine cracks above the reinforcement due to formwork settlement, to wide cracks due to foundation or support settlement.

Figure 2.2.3.2c – Crack/separation in abutment wall due to differential settlement

Figure 2.2.3.2d – Shear crack in girder adjacent to support

2.2.3.3 Significance of cracks

Crack type and widths provide some insight into how the cracks affect structure durability and strength. The presence of cracks can provide access to moisture and oxygen and the exchange of aggressive substances beyond cover concrete to embedded steel, promoting corrosion and further deterioration through cracking, spalling and delamination.

The aggressiveness of exposure conditions will directly influence the likelihood of further deterioration from cracks. For example, in a marine environment subject to wetting and drying, the presence of cracks will likely result in severe deterioration over a relatively short period.

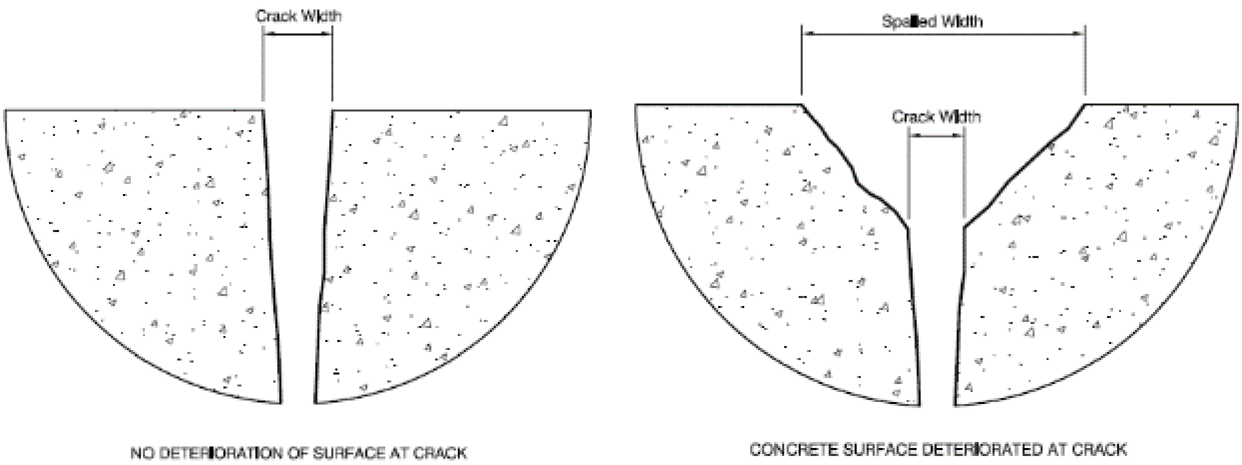
The location and orientation of cracks is usually a good indicator of the likely causes.

Another key factor in understanding the significance of cracks is whether they are passive or active. Passive cracks will be dimensionally stable whereas active cracks continue to grow. This emphasises the importance of measuring crack widths accurately. Over time, the concrete surface around a crack deteriorates and spalling of the crack edges will occur. It is important to ensure that the actual crack width, and not the spalled width is measured (refer Figure 2.2.3.3).

The severity of cracking is reported as a function of the crack width:

Hairline	up to 0.1 mm
Minor	0.1 to 0.3 mm
Moderate	0.3 to 0.6 mm
Severe	Greater than 0.6 mm.

Figure 2.2.3.3 – Accurate measurement of crack width



2.2.4 Spalling

A spall is a fragment of concrete which has been detached from a larger concrete mass. The causes of spalling include:

- Corrosion - a continuation of the corrosion process whereby the actions of external loads or pressure exerted by the corrosion of reinforcement and attendant expansion results in the breaking off of the delaminated concrete. The spalled area left behind is characterised by sharp edges.
- Impact - vehicular or other impact forces on exposed concrete edges, deck joints or construction joints can result in the spalling or breaking off of pieces of concrete locally.
- Compression - overloading of concrete in compression can result in the breaking off of the concrete cover to the depth of the outer layer of reinforcement. Spalling may also occur in areas of localised high compressive load concentrations, such as at structure supports, or at anchorage zones in prestressed concrete.
- External restraint/forces – e.g. restraint forces generated by seized bearings often cause spalling of the bearing support area on the front face of the bearing shelf.

- Fire – spalling of concrete may result in concrete exposed to extreme temperatures such as fire.

Examples of concrete spalling are shown in Figure 2.2.4a to Figure 2.2.4d.

Figure 2.2.4a – Spalling of concrete at ends of deck units

Figure 2.2.4b – Spalling to front face of headstock

Figure 2.2.4c – Spalling of slab soffit

Figure 2.2.4d – Corner spalling of square pile

2.2.5 Delamination

Delamination is defined as a discontinuity in the surface concrete which is substantially separated but not completely detached from concrete below or above it. Visibly, it may appear as a solid surface but can be identified as a hollow sound by tapping. Delamination begins with the corrosion of reinforcement and subsequent cracking of the concrete. However, in the case of closely spaced bars, the cracking extends in the plane of the reinforcement parallel to the exterior surface of the concrete (refer Figure 2.2.6a).

2.2.6 Surface defects

Surface defects are not necessarily serious in themselves however they are indicative of a potential weakness in concrete. Surface defects include:

- Segregation - the differential concentration of the components of mixed concrete resulting in non-uniform proportions in the mass. Segregation is caused by concrete falling from a height, with coarse aggregates settling to the bottom and the fines on top. Another form of segregation occurs where reinforcing bars prevent the uniform flow of concrete between them. Segregation is more likely to occur in higher slump concrete.
- Cold joints – these are produced if there is a delay between placement of successive pours of concrete, and if an incomplete bond develops at the joint due to the partial setting of concrete in the first pour (refer Figure 2.2.6b).

Figure 2.2.6a – Delamination in culvert slab

Figure 2.2.6b – Cold joints in abutment wall

- Deposits – staining/accumulation of material on concrete surfaces where water percolates through the concrete and dissolves or leaches chemicals from it and deposits them on the surface. Deposits may appear as:
  - efflorescence – a deposit of salts, usually white. Over time can result in stalactites forming (refer Figure 2.2.6c)
  - exudation – a liquid or gel-like discharge through pores or cracks in the surface (refer Figure 2.2.6d)
  - encrustation - a hard crust or coating formed on the concrete surface.

Figure 2.2.6c – Efflorescence evident on concrete headstock

Figure 2.2.6d – Discharge through cracks in concrete surface

- Honeycombing – produced due to the improper or incomplete vibration of the concrete which results in voids being left in the concrete where the mortar failed to completely fill the spaces between the coarse aggregate particles (refer Figure 2.2.6e).
- Abrasion - the deterioration of concrete brought about by vehicles scraping against concrete surfaces, such as decks, kerbs, barrier walls, piers or the result of dynamic and/or frictional forces generated by vehicular traffic, coupled with abrasive influx of sand, dirt and debris. It can also result from friction of waterborne particles against partly or completely submerged members (refer Figure 2.2.6f). This phenomenon is also known as water wash.
- Polishing - a slippery surface may result from the polishing of the concrete deck surface by the action of repetitive vehicular traffic where inadequate materials and processes have been used.

Figure 2.2.6e – Concrete honeycombing

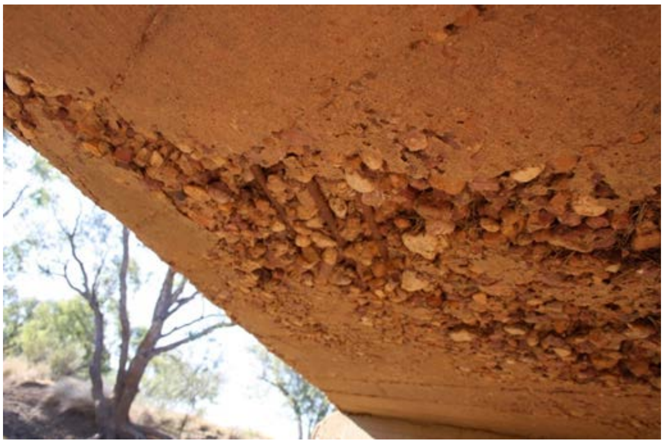


Figure 2.2.6f – Abrasion of concrete

2.2.7 Scaling

Scaling is the local flaking or loss of the surface portion of concrete or mortar. Scaling is common in non air-entrained concrete but can also occur in air-entrained concrete in the fully saturated condition. Scaling occurs in poorly finished or overworked concrete where too many fines and not enough entrained air is found near the surface. Scaling of concrete is shown in Figure 2.2.7a.

Figure 2.2.7a – Concrete scaling

Figure 2.2.7b – Concrete disintegration

2.2.8 Disintegration

Disintegration is the physical deterioration or breaking down of the concrete into small fragments or particles. The deterioration usually starts in the form of scaling and, if allowed to progress beyond the level of very severe scaling, is considered as disintegration. Disintegration of concrete is illustrated in Figure 2.2.7b.

2.2.9 Fire

Concrete structures can sustain damage when exposed to fire depending on the duration and intensity. The effects of high temperature fires on concrete structures can include:

- reduction in compressive strength
- reduction in modulus of elasticity
- micro-cracking within concrete matrix
- spalling of concrete
- loss of bond between steel and concrete
- possible loss of residual strength of steel reinforcement and/or loss of tension in prestressing tendons.

2.2.9.1 Effects on concrete

Common changes in concrete properties associated with various peak temperatures are summarised below:

- Up to 120ºC – no significant effects.
- Up to 250ºC – localised cracking and dehydration of cementitious paste, complete loss of free moisture. Commencement of strength reduction.
- 300 – 600ºC – significant cracking of cementitious paste and aggregates due to expansion. Colour of concrete changes to pink.
- 600ºC – complete dehydration of cementitious paste with associated shrinkage cracking and honey combing. Concrete becomes friable, very porous and easily broken down. Colour of concrete changes to grey. Strength lost.

- 1200ºC – constituent components start to melt.
- 1400ºC – concrete melts completely.

2.2.9.2 Effects on reinforcing steel

Steel reinforcement can exhibit up to 50% loss in yield strength while at elevated temperatures of around 600ºC. Recovery of yield strength will typically occur for temperatures up to 450ºC for cold worked steel products and up to 600ºC for hot rolled steel products. For temperatures beyond these ranges the loss in yield strength is permanent. The modulus of elasticity is also reduced while the steel is at elevated temperatures.

Pre-stressing steel is more susceptible to the effects of fire and elevated temperatures because loss of strength in the order of 50% occurs at temperatures of about 400ºC. Loss of tension in tendons occurs due to a combination of the elevated temperature effects and loss of modulus of elasticity of the concrete.

The bond between steel and concrete can be adversely affected at temperatures greater than 300ºC (because of the difference in thermal conductivity and thermal expansion properties between the steel and cover concrete).

2.3 Steel

The use of steel has progressed from cast iron, wrought iron, rivet steel and plain carbon steel to notch tough low temperature steel.

The following defects/issues with steel are described:

- corrosion
- protective coating systems
- permanent deformations
- cracking
- loose connections
- impact damage
- fire damage.

2.3.1 Corrosion

Corrosion is the deterioration of steel by chemical or electro-chemical reaction resulting from exposure to air, moisture, industrial fumes and other chemicals and containments in the environment in which it is placed. The terms rust and corrosion are used interchangeably in this sense. Corrosion, or rusting, will only occur if the steel is not protected or if the protective coating wears or breaks down.

Figure 2.3.1a – Mild surface corrosion

Figure 2.3.1b – Severe corrosion and loss of section

All ferrous alloys are susceptible to corrosion deterioration. Unprotected steel, in the presence of oxygen and moisture and in the absence of contaminants (clean atmospheric conditions) corrodes at approximately 0.02 mm/year. The rate of deterioration is therefore very slow and it will be many years before the integrity of a structure is compromised. However, corrosion is accelerated by continuous (or even intermittent) wet conditions or by exposure to aggressive ions, such as chlorides in de-icing salts or in a marine environment, and other atmospheric industrial contaminants. In these conditions, steel becomes vulnerable to both pitting and general corrosion (refer Figure 2.3.1a and Figure 2.3.1b).

Pitting corrosion is a local large reduction in parent metal and can cause a serious reduction in load-carrying capacity (refer Figure 2.3.1c). It can also lead to local high stresses, which may increase the risk of fatigue failure.

Other causes of steel corrosion are the flux used in welding (if not neutralised), and direct contact with dissimilar metals, but, with the exception of bi-metal contact, these are less likely to cause significant structural deterioration.

Rust on carbon steel is initially fine grained, but as rusting progresses it becomes flaky and delaminates exposing a pitted surface. The process thus continues with progressive loss of section.

Sulphate Reducing Bacteria (SRB) is a corrosion mechanism of galvanised steel components in contact with soil water. The reduction of the sulphate content on the steel surface results in a highly acidic environment within and under the affected area which causes steel corrosion (refer Figure 2.3.1d). Favourable conditions for SRB growth include oxygen-depleted, marine or freshwater pervious soils (sediments) containing sulphate at a neutral pH. Further information relating to this phenomenon can be found in Technical Note 99 *Sulphate Reducing Bacteria on Steel Structures*.

Figure 2.3.1c – Pitting of steel

Figure 2.3.1d – Corrosion of steel by SRB

2.3.2 Protective coating systems

Structural steelwork is normally protected against corrosion by a protective coating system (paint or galvanising). Weathering steel, which does not require a protective coating, is the only exception.

Corrosion of steelwork is usually associated with the breakdown or inadequacy of the protective system.

Paint systems suffer from various forms of deterioration such as cracking, flaking, chalking and peeling. The life of a paint system is normally much less than that of the steel member it is protecting. Early detection of breakdown is beneficial because it substantially reduces the amount of preparation that is involved in repair and reapplication. Delays to the maintenance of paint systems can result in rapidly accelerating increased costs.

Maintenance of steelwork protected by galvanising becomes extremely difficult if remedial action is delayed until corrosion is well established.

The cause of any white deposit on the surface of paint over zinc metal spray should be investigated as it may be zinc hydroxide (formed by reaction of zinc with water and air), which is the first sign of the zinc coating breaking down. If left untreated corrosion of the zinc will become extensive (refer Figure 2.3.2.a).

Aluminium metal spray is less easily attacked, breakdown usually occurs because the aluminium spray has been badly applied.

Breakdown of paint over galvanising is often due to the poor adhesion of a wrongly selected paint system (refer Figure 2.3.2b).

Figure 2.3.2a – White deposit on zinc metal spray

Figure 2.3.2b – Failure of paint system over galvanising

Common types of protective system failure are:

- Blistering - generally caused either by solvents which are trapped within or under the paint film, or, by water which is drawn through the paint film by the osmotic forces exerted by hygroscopic or water soluble salts at the paint/substrate interface. The gas or the liquid then exerts a pressure stronger than the adhesion of the paint (refer Figure 2.3.2c).

Figure 2.3.2c – Blistering of paint system

Figure 2.3.2d – Flaking of paint system

- Corrosion blistering - coatings generally fail by disruption of the paint film by expansive corrosion products at the coating/metal interface. General failure can result from inadequate paint film thickness. However, local or general deterioration can occur when corrosion is due to water and aggressive ions being drawn through the film by the osmotic action of soluble iron corrosion products, as the attack will start from corrosion pits.
- Flaking - flaking or loss of adhesion is generally visible as paint lifting from the underlying surface in the form of flakes or scales (refer Figure 2.3.2d). If the adhesive strength of the film is strong then the coating may form large shallow blisters. Causes include:
  - Loose, friable or powdery materials on the surface before painting.
  - Contamination preventing the paint from ‘wetting’ the surface, i.e. oil, grease, etc.
  - Surface too smooth to provide mechanical bonding.
  - Application of materials in excess of their pot life.

- Chalking - the formation of a friable, powdery coating on the surface of a paint film caused by disintegration of the binder due to the effect of weathering, particularly exposure to sunlight and condensation. This is generally considered the most acceptable form of failure since maintenance surface preparation consists only of removing loose powdery material and it is usually unnecessary to blast clean to substrate.
- Cracking - may be visible in increasing extent, ranging from fine cracks in the top-coat to deeper and broader cracks.
- Pinholes - minute holes formed in a paint film during application and drying. They are caused by air or gas bubbles (perhaps from a porous substrate such as metal spray coatings or zinc silicates) which burst, forming small craters in the wet paint film which fail to flow out before the paint has set.

2.3.2.1 Weathering steel

Weathering steel owes its corrosion protection to the formation of a stable protective oxide film, which seals the surface against further corrosion. The film is dark brown with a lightly textured 'rusty' surface. Unlike other types of structural steel, weathering steel has a carefully controlled non-ferrous content to ensure that the oxide film adheres tightly to the substrate. Nevertheless, the surface oxide is slowly worn away, and replaced by a new film, causing a very slow loss of section over the life of the structure. The rate of loss depends on the alloy content, air quality and the frequency with which the surface is wetted by dew and rainfall and dried by the wind and sun. Designs using weathering steels typically allow for a 1 mm to 2 mm sacrificial loss to sections and fasteners. The location/environment for the use of weathering steel must be carefully considered in terms of wet climate conditions. Special attention needs to be paid to management of water run-off and drainage.

2.3.3 Permanent deformations

Permanent deformation or distortion of steel members can take the form of bending, buckling, twisting or elongation, or any combination of these.

Permanent deformations may be caused by initial distortion, residual stresses, lack of fit, inadequate design, overloading, impact damage, or inadequate or damaged intermediate lateral supports or bracing (refer to Figure 2.3.3a and Figure 2.3.3b).

Permanent bending deformation may occur in the direction of the applied loads and are usually associated with flexural members; however, vehicular impact may produce permanent deformations in bending in any other member.

Figure 2.3.3a – Buckling of gusset plate

Figure 2.3.3b – Impact damage to steel beam

Permanent buckling deformations normally occur in a direction perpendicular to the applied load and are usually associated with compression members. Buckling may also produce local permanent deformations of webs and flanges of beams, plate girders and box girders.

Permanent twisting deformations appear as a rotation of the member about its longitudinal axis and are usually the result of eccentric transverse loads on the member. Permanent axial deformations occur along the length of the member and are normally associated with applied tension loads.

Any distortion of a member out of plane in the form of waves, kinks or warping can significantly reduce resistance to compressive load.

2.3.4 Cracking

A crack is a linear fracture of the steel component.

Cracks are potential causes of complete fracture and usually occur at connections and changes in section. The most common causes are fatigue and poor detailing practices that produce high stress concentrations. Elements that have been modified since initial construction are also potential problem areas. Fracture of any member, bolt, rivet or weld is obviously serious and can have important structural implications.

Fatigue failure is the most common cause of cracking and fracture of steelwork structures. Fatigue is the process by which a structural member or element eventually fails after repeated applications of cyclic stress. Failure may occur even though the maximum stress in any one cycle is considerately less than the fracture stress of the material. Characteristically, a fatigue fractured surface displays two distinct zones: a smooth portion indicating stages in the growth of the fatigue crack, and a rough surface, which represents the final ductile tearing or cleaving. Typically, fatigue failures do not exhibit any significant ductile ‘necking’ and occur without prior warning or plastic deformation (refer Figure 2.3.4a and Figure 2.3.4b).

Figure 2.3.4a – Fatigue cracking in web

Figure 2.3.4b – Cracking in welds

The risk of fatigue-induced failure may exist in bridges:

- not designed for fatigue
- not designed to adequate fatigue criteria
- where materials and manufacturing controls may not have been adequate
- where structural changes may have occurred, which may include the addition of new fixtures or repair of damage using, for example, welded cleats or brackets, flame cut holes or strengthening plates
- where operational changes have occurred, such as alterations to carriageway layouts and/or increased vehicle loading
- where there is evidence of resonance occurring in any of the structural members.

Fatigue starts with fabrication flaws or at locations with high surface stress concentrations such as weld toes, irregular cut edges and flame cut edges. It then proceeds through the growth of these flaws until a final failure mode, such as brittle fracture or buckling, occurs. The initial fabrication flaws may be large or small but in many cases are too small to be detected by eye.

Cracks may also be present in welds because of poor welding techniques or inappropriate materials. If cracks are detected it is likely that they will be repeated in similar details within the structure.

Cracks may also be caused or aggravated by overloading, vehicular collision or loss of section resistance due to corrosion. In addition, stress concentrations due to the poor quality of the fabricated details and the fracture toughness of materials used are contributing factors. Material fracture toughness will determine the size of the crack that can be tolerated before fracture occurs.

Welded details are more prone to cracking than bolted or riveted details. Grinding off the weld reinforcement to be smooth or flush with the joined metal surfaces improves fatigue resistance. Once the cracking occurs in a welded connection, it can extend into other components due to a continuous path provided at the welded connection, and possibly lead to a brittle fracture.

Bolted or riveted connections may also develop fatigue cracking, but a crack in one component will generally not pass through into the others. Bolted and riveted connections are also susceptible to cracking or tearing resulting from prying action, and by a build-up of corrosion forces between parts of the connection.

Common locations susceptible to cracking are illustrated in Figure 2.3.4c and Figure 2.3.4d. As cracks may be concealed by rust, dirt or debris, the suspect surfaces should be cleaned prior to inspection.

Cracks that are perpendicular to the direction of stress are very serious, with those parallel to the direction of stress less so. In either case, cracks in steel should generally be considered serious, as a parallel crack may for a number of reasons turn into a perpendicular crack. Any crack should be carefully noted and recorded as to its specific location in the member, and member structure. The length, width (if possible) and direction of crack should also be recorded.

Figure 2.3.4c – Common crack locations in steel

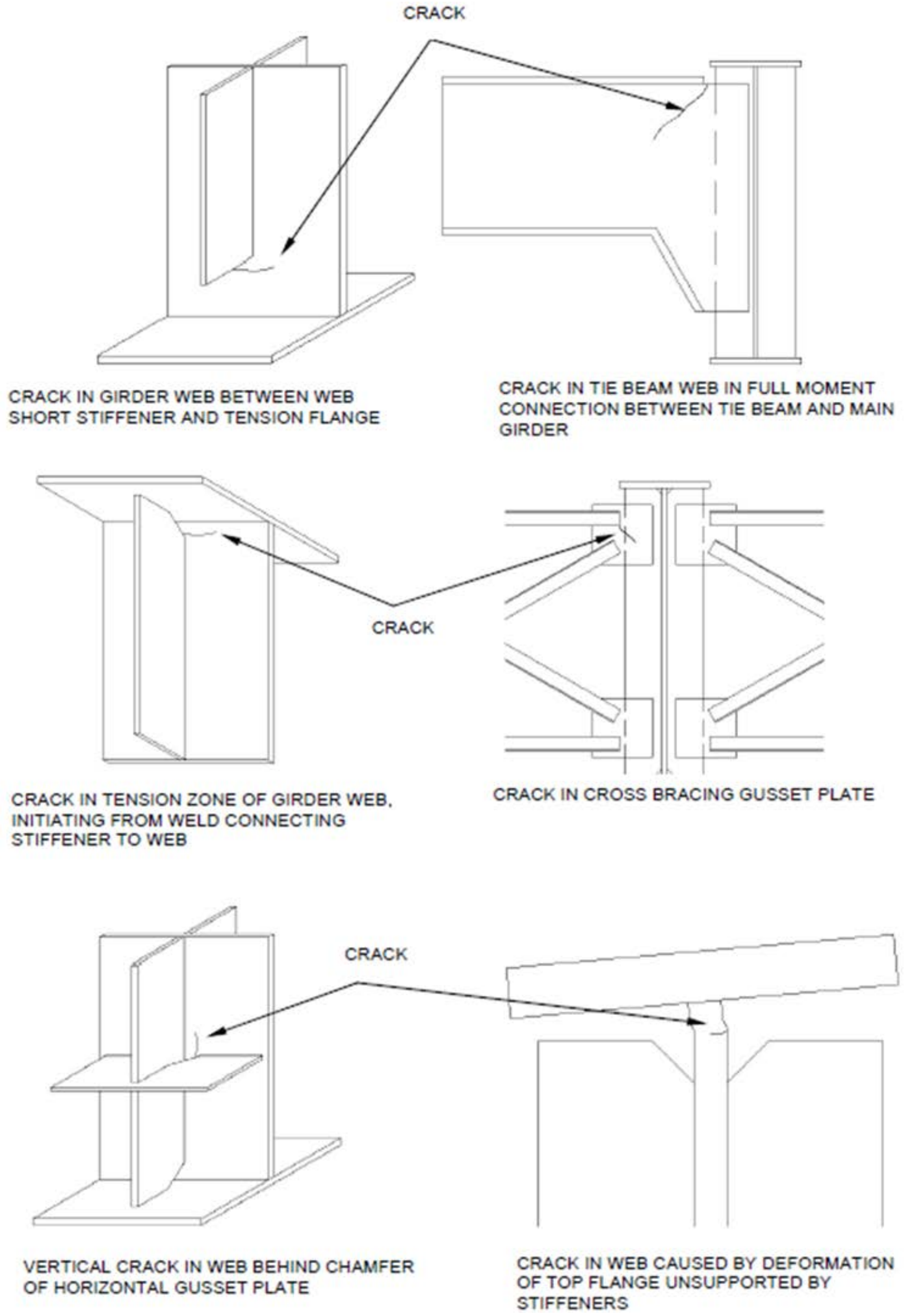
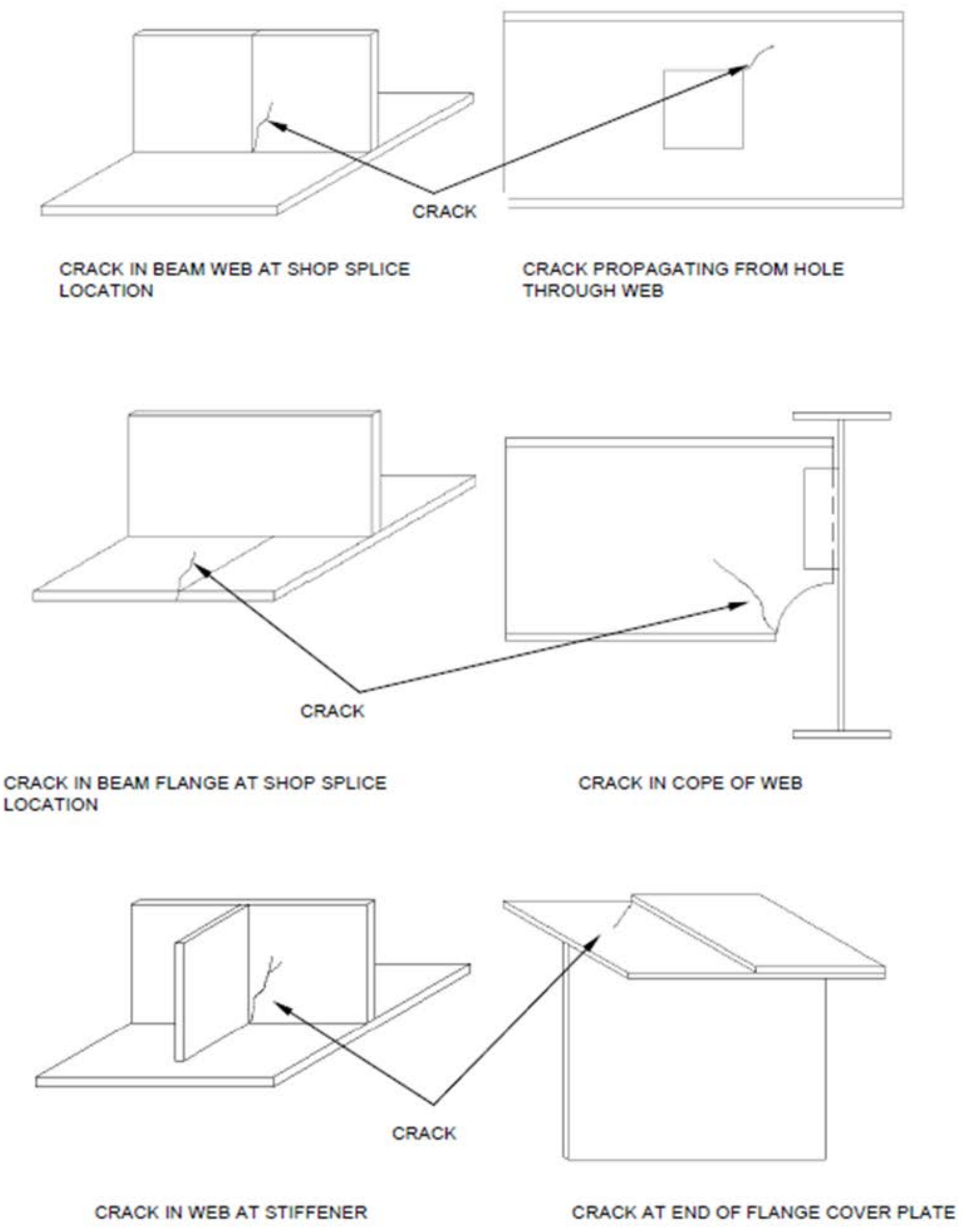


Figure 2.3.4d – Common crack locations in steel



2.3.5 Loose connections

Loose connections can occur in bolted or riveted connections; and, may be caused by incorrect installation, corrosion of the connector plates or fasteners, excessive vibration, overstressing, cracking or the failure of individual fasteners.

Loose connections may sometimes not be detectable by visual inspection. Cracking or excessive corrosion of the connector plates or fasteners, or permanent deformation of the connection or members framing into it, may be indications of a loose connection. Tapping the connection with a hammer is one method of determining if the connection is loose.

2.3.6 Fire damage

Steel progressively weakens with increasing temperature, e.g. the yield strength at room temperature is reduced by about 50% at 550°C, and to about 10% at 1000°C. There is therefore a risk that steel members may fail by buckling or deflecting if they are inadvertently heated during a traffic accident fire. The extent of failure will depend on the loading that the member is carrying, its support conditions, and the temperature gradient through the cross section.

Secondary effect damage can occur in bearings, movement joints and other structural members if they are unable to accommodate the large expansions that may occur in a fire. It is unlikely that this will have been allowed for in design.

In a severe fire unprotected steelwork will lose practically all its load bearing capacity, deform and distort and will not be suitable for reuse. In a less severe fire, damage may be limited and it may be possible to retain members after checking for straightness and distortion and the mechanical properties. Bolted connections often fail through shear or tensile failure or thread stripping. Any section yielding could have caused severe weakening of connections and it is important that these are properly inspected.

Fire will also cause blistering and flaking of paintwork.

2.3.7 Impact damage

Impact damage to a steel structure is usually obvious and will vary in significance from abrasion of the protective coating through to deformation of a component. In severe cases the load carrying capacity of a component may be compromised.

2.4 Timber

Timber was extensively used for bridges constructed up until the middle 1900's and these constitute 10% of the structures on the State Declared Road network. The largest proportion of timber bridges occurs on roads of lesser importance such as local roads, but many timber bridges are still in service on higher class roads and are often required to carry heavy traffic loadings.

The major causes of deterioration in timber bridges are as follows:

i. fungi (rotting)
ii. termites
iii. marine organisms
iv. corrosion of fasteners
v. shrinkage and splitting
vi. fire damage
vii. weathering
viii. flood damage.

2.4.1 Fungi

Severe internal decay of timbers used for bridges is caused mainly by ‘white rot’ or ‘brown rot fungi. External surface decay, especially in ground contact areas, is caused by ‘soft rot’ fungi. Other fungi such as mould and sapstain fungi may produce superficial discolorations on timbers but are not generally of structural significance.

Fungal growths will not develop unless there is a source of infection from which the plants can grow. Fungi procreate by producing vast numbers of microscopic spores which may float through the air for long periods and can be blown for considerable distances.

Although it is true to say that no timber components in service will be free from decay because of an absence of infecting spores, these spores will not germinate and develop unless there is:

- an adequate supply of food (wood cells).
- an adequate supply of air or oxygen (prolonged immersion in water saturates timber and inhibits fungal growth)
- a suitable range of temperatures (optimum temperatures are 20ºC – 25ºC for soft rots, while their rate of growth declines above or below the optimum with a greater tolerance of lower temperatures apparent)
- a continuing supply of moisture (wood with a moisture content below 20% is considered safe from decay, while many fungi require a moisture content above 30%).

Figure 2.4.1a – Fungal fruiting body and decay of girder

Figure 2.4.1b – Decay pocket in girder

Once established, the decay fungi continue to grow at an accelerating rate as long as favourable conditions prevail. Depriving the fungi of any one of these required conditions will effectively curtail the spread of decay. Wood that is kept dry or saturated will not rot. Moisture change can affect decay indirectly because drying often leads to surface checks, which may expose untreated parts of timber or create water trapping pockets. Proper preservative treatment effectively provides a toxic barrier to the fungi's food supply, thus preventing decay.

Figure 2.4.1a and Figure 2.4.1b show rotting of members. The most common rotting areas in timber bridges are internally in log girders, corbels, headstocks and piles (piping), and in sawn decking at the exposed ends and interface with kerbs. Decay is often more pronounced at the ends of members.

2.4.2 Termites

Australia has a large number of different species of termites (300) which are widely distributed. Practically all termite damage to timber bridges occurs through subterranean termites (especially Coptotermes acinaciformis and allied species) which require contact with the soil or some other constant source of moisture. Some dry wood termite species are found in coastal areas of Queensland, but minimal damage is attributable to these types in bridges.

Termites live in colonies or nests which may be located below ground in the soil, or above ground in a tree stump, hollowed out bridge member or an earth mound. Each colony contains a queen, workers, soldiers and reproductives or alates. The workers, who usually constitute the highest portion of the population, are white bodied, blind insects some 3 mm in length which have well developed jaws for eating timber. However, in North Queensland, termites growing to 20 mm or more in length (Mastotermes darwiniensis) are found and these are capable of causing significant damage in a short time compared to the most commonly distributed species.

Attack by subterranean termites originates from the nest, but may spread well above ground level, either inside the wood or via mud walled tubes called galleries which are constructed on the outside of bridge members (refer Figure 2.4.2a and Figure 2.4.2b). These galleries are essential for termites as they require an absence of light, a humid atmosphere and a source of moisture to survive. At least once a year the alates develop eyes and wings and leave the nest under favourable weather conditions to migrate up to 200 m from the original nest. After migration, their wings fall off and a few of these may pair to start new colonies.

Figure 2.4.2a – Termite nest on bridge pile

Figure 2.4.2b – Termite gallery on pier headstock

Termite attack, once established, usually degrades timber much more quickly than fungi, but termite attacks in durable hardwoods normally used in bridge construction is usually associated with some pre-existing fungal decay. This decay accelerates as the termites extend their galleries through the structure, moving fungal spores and moisture about with their bodies. Hence, although some of the material removed by termites has already lost structural strength because of decay, the control of termites remains an extremely important consideration.

Basically, there are two main strategies in termite control:

- eradication of the nest (by either direct chemical treatment or by separation of the colony from its sustaining moisture)
- installation of chemical and physical barriers to prevent termites from entering a bridge or attacking timber in contact with the ground.

In practice it may be difficult to eradicate the nest because of the problem of locating it.

2.4.3 Marine organisms

Damage to underwater timber in the sea or tidal inlets is usually caused by marine borers, and is more severe in tropical and sub-tropical waters than in colder waters.

The two main groups of animal involved are:

- Molluscs (teredinindae) - this group includes various species of Teredo, Nausitora and bankia. They are commonly known in Australia as teredo or "shipworm". They start life as minute, free-swimming organisms and after lodging on timber they quickly develop into a new form and commence tunnelling. A pair of boring shells on the head grow rapidly in size as the boring progresses, while the tail with its two water circulating syphons remains at the original entrance. The teredine borers destroy timber at all levels from the mudline to high water level, but the greatest intensity of the attack seems to occur in the zone between 300 mm above and 600 mm below tide level. A serious feature of their attack is that while the interior of the pile may be eaten away, only a few small holes may be visible on the surface (refer Figure 2.4.3a).
- Crustaceans - this group includes species of Sphaeroma (pill bugs), Limnoria (gribbles), and Chelura. They attack the wood on its surface, making many narrower and shorter tunnels than those made by the teredines. The timber so affected is steadily eroded from the outside by wave action and the piles assume a wasted appearance or "hourglass effect" (refer Figure 2.4.3b). Attack by Sphaemora is limited to the zone between tidal limits, with the greatest damage close to half-tide level. They cannot survive in water containing less than 1.0 - 1.5 percent salinity, but can grow at lower temperatures than the teredines.

Figure 2.4.3a – External damage to timber pile from shipworm

Figure 2.4.3b – Timber piles subject to Sphaemora attack

Many strategies have been developed for the control of marine borers but, assuming that the piles have sufficient remaining strength, the most effective work by reducing the oxygen content of water around the borers.

2.4.4 Corrosion of fasteners

Corrosion of steel fasteners can cause serious strength reductions for two related reasons. Firstly, the steel fastener reduces in size and weakens, and secondly a chemical reaction involving iron salts from the rusting process can significantly reduce the strength of the surrounding wood (this is not fungal decay but may enhance corrosion of the fastener because of water ingress in the softened timber).

Galvanised fasteners in contact with timber which has been freshly treated with CCA preservative may exhibit enhanced corrosion. However, for CCA treated timber that has been cured for six weeks, normal corrosion rates for fasteners will apply.

2.4.4.1 Shrinkage and splitting

Moisture can exist in wood as water or water vapour in the cell cavities and as chemically bound water within the cell walls. As green timber loses moisture to the surrounding atmosphere, a point is reached when the cell cavities no longer contain moisture, but the cell walls are still completely saturated with chemically bound water. This point is called the ‘fibre saturation point’. Wood is dimensionally stable while its moisture content remains above the fibre saturation point, which is typically around 30% for most timbers. Bridges are normally constructed from green timber which gradually dries below its fibre saturation point until it reaches equilibrium with the surrounding atmosphere. As it does so, the wood shrinks but because it is anisotropic, it does not shrink equally in all directions. Maximum shrinkage occurs parallel to the annular rings, about half as much occurs perpendicular to the annular rings and a small amount along the grain.

Figure 2.4.4.1a – Splitting in timber pile

Figure 2.4.4.1b – Splitting in timber headstock

The relatively large cross section timbers used in bridges lose their moisture through their exterior surfaces so that the interior of the member remains above the fibre saturation point while the outer layers fall below and attempt to shrink. This sets up tensile stresses perpendicular to the grain and when these exceed the tensile strength of the wood, a check or split develops, which deepens as the moisture content continues to drop. As timber dries more rapidly through the ends of the member than through the sides, more serious splitting occurs at the ends. Deep checks provide a convenient site for the start of fungal decay. Figure 2.4.4.1a and Figure 2.4.4.1b show longitudinal splitting of timber headstocks and piles.

Shrinkage also causes splitting where the timber is restrained by a bolted steel plate or other type of fastening. This splitting can be avoided by allowing the timber to shrink freely by using slotted holes. As timber shrinks, it tends to lose contact with steel washers or plates, so the connection is no longer tight. Checking the tightness of nuts in bolted connections is therefore a standard item of routine maintenance for timber bridges.

2.4.5 Fire

Wood itself does not burn. The effect of heat is firstly to decompose the wood (a process known as 'pyrolysis') and it is some of the products of this decomposition that burn if conditions are suitable. This concept is important in discussions on the action of retardants.

In theory, wood decomposes even at temperatures as low as 20ºC (at the rate of 1% per century). At 93ºC the wood will become charred in about five years.

When wood is heated, several zones of pyrolysis occur which are well delineated due to the excellent insulating properties of wood (thermal conductivity roughly 1/300 that of steel). These zones can be described generally as follows:

- Zone A: 95ºC – 200ºC.
- Water vapour is given off and wood eventually becomes charred.
- Zone B: 200ºC – 280ºC.
- Water vapour, formic and acetic acids and glyoxal are given off, ignition is possible but difficult.
- Zone C: 280ºC – 500ºC.

Combustible gases (carbon monoxide, methane, formaldehyde, formic and acetic acids, methanol, hydrogen) diluted with carbon dioxide and water vapour are given off. Residue is black fibrous char. Normally vigorous flaming occurs. If, however, the temperature is held below 500ºC, a thick layer of char builds up and because the thermal conductivity of char is only 1/4 that of wood, it retards the penetration of heat and thus reduces the flaming.

- Zone D: 500ºC – 1000ºC

In this zone the char develops the crystalline structure of graphite, glowing occurs and the char is gradually consumed.

- Zone E: above 1000ºC

At these temperatures the char is consumed as fast as it is formed.

As the temperature of the wood is lowered, the above mentioned behaviour still holds, e.g., combustion normally ceases below 280ºC.

Large section round timbers, as used in bridge construction, have good resistance to fire, and, except during a severe bush fire, usually survive quite successfully.

2.4.6 Weathering

Weathering is the gradual deterioration of sawn or log timber due to its exposure to sun, wind and rain. Weathering can be a serious problem especially to the exposed end grain of untreated or unprotected wood, where severe rotting can occur around the connections. The exposed ends of transverse deck planks are susceptible to this defect.

2.4.7 Floods

Floods can have a disastrous affect particularly on timber structures. This is due to:

- extra pressure from the flood waters and debris
- log impact on the substructure. If the flood is high enough, the super-structure can also be damaged by the flood waters.

2.5 Masonry

Masonry construction comprises individual stones, bricks or blocks bonded together by mortar. Although not a common construction material today, masonry has been used in retaining walls, abutments, piers and arches. Types of masonry construction are Ashlar masonry, squared stone masonry and rubble masonry.

The following defects commonly occurring in masonry are described:

i. cracking

ii. splitting, spalling and disintegration

iii. loss of mortar and stones.

2.5.1 Cracking

A crack is an incomplete separation into one or more parts with or without space in between. Cracks develop in masonry as a result on non-uniform settlement of the structure, thermal restraint and overloads.

Cracks develop either at the interface between the stone and mortar, following a zigzag pattern, when the bond between them is weak; or, go through the joint and stone in a straight line, when the mortar is stronger than the stone.

2.5.2 Splitting, spalling and disintegration

Splitting is the opening of seams or cracks in the stone leading to the breaking of the stone into large fragments.

Spalling is the breaking or chipping away of pieces of the stone from a larger stone.

Disintegration is the gradual breakdown of the stone into small fragments, pieces or particles.

The splitting, spalling and disintegration of masonry is caused by the actions of weathering and abrasion; or, by the actions of acids, sulphates or chlorides, which cause deterioration in certain types of stones, such as limestone.

2.5.3 Loss of mortar and stones

Loss of mortar is the result of the destructive actions of water wash, plant growth or softening by water containing dissolved sulphates or chlorides. Once the mortar has disintegrated it may lead to loss of stones. Excessive loss of mortar will also reduce the load-carrying capacity of a structure.

2.5.4 Arch stones dropping

Adopted from *VicRoads Road Structures Inspection Manual* (Ref. 12):

> *Ground or foundation movement or severe vibration can cause stone blocks to displace and drop relative to other stones in an arch. This can also be exacerbated if the quality of the stones or mortar is poor and failing.*

2.5.5 Deformation

Adopted from *VicRoads Road Structures Inspection Manual* (Ref. 12):

> *Arches are either semi-circular, segmental (i.e. part of a semi-circle) or elliptical in shape. The regular curvature may become deformed if the arch is overloaded or if there is differential settlement of the foundations. Deformation may be accompanied by cracking and dropped stones. The position and degree of deformation should be recorded.*

2.6 Fibre Reinforced Polymers (FRP)

2.6.1 FRP strengthening

Fibre Reinforced Polymer (FRP) composites are used to strengthen reinforced and prestressed concrete members which are deficient in moment, shear or bursting capacity. The fibres can be Carbon, Aramid or Glass. The FRP material can be used in the form of flexible sheets to wrap around the member or in the form of plates. Plates comprise one of the three fibre types, typically in a resin or epoxy matrix. The system relies on the high tensile capacity of FRP and the bond between the FRP and the steel or concrete beam.

FRP strengthening can be detrimentally affected by overloading of the structures, extreme temperature, moisture absorption and high UV exposure. The effects are exacerbated by defects introduced in the materials during manufacture, handling and installation. The strengthening method relies entirely on the anchorage and bond of the FRP material to the base component.

The following areas should be inspected and recorded:

- The ends of the strengthened area for signs of the FRP strips debonding from the epoxy resin or the resin debonding from the concrete base.
- The visible concrete surface at the edge of the strengthening for signs of cracking or spalling which could affect bonding between the FRP and the member.
- The whole of FRP surface for signs of delamination from the concrete or any irregularities in the material such as blistering or folding.
- Tears, cuts or crazing of the FRP material.

If any area is classified as being in condition states 3 or 4, pull-off testing should be conducted in the surrounding FRP to ensure the full extent of the problem is identified. Repair should not be instigated until the whole area of the defect has been identified.

2.6.2 FRP components

In addition to strengthening of existing components, FRP girders are increasingly being used to replace hardwood timber components at end of life and have also been trialled for construction of new superstructures.

Two systems currently in use on the network for component replacement are the Wagners fibre composite girder (Wagners FC) and the Loc Composites developed fibre composite hybrid girder.

The Wagners FC girder comprises FRP pultrusions, steel reinforcement and exterior flow coat (refer Figure 2.6.2a and Figure 2.6.2c).

Loc Composites' fibre composite girder comprises chemically treated plantation softwood Laminated Veneer Lumber (LVL) with several glass pultrusions/steel modules incorporated into the section (refer Figure 2.6.2b and Figure 2.6.2d).

Figure 2.6.2a – Wagners FC girder (1st generation)

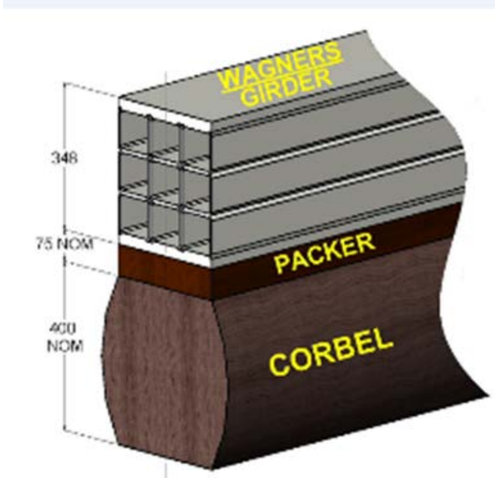


Figure 2.6.2b – Loc Composite Hybrid girder (1st generation)

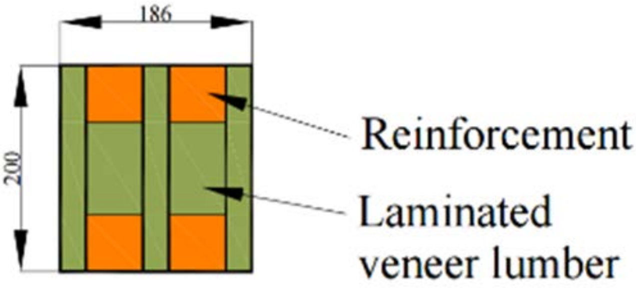


Figure 2.6.2c – Wagners FC girder (2nd generation – WCFT S1, S2 & S3)

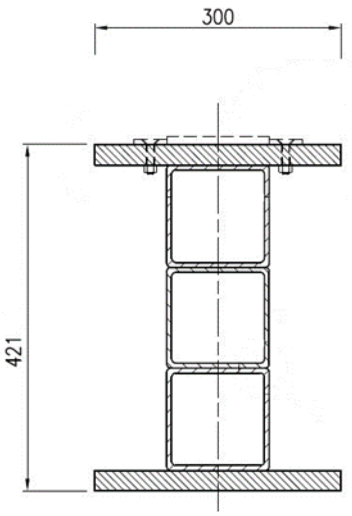
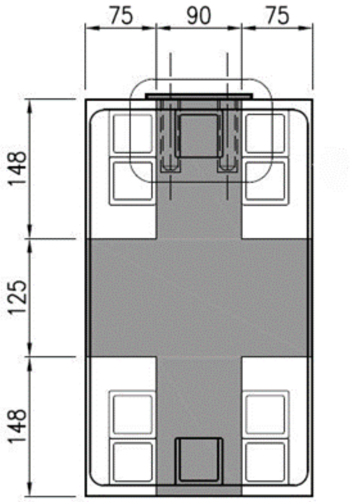


Figure 2.6.2d – Loc Composite Hybrid girder (2nd generation -LOC 400 & LOC 420))



Areas of concern relating to durability/deterioration of FRP components align with the constituent materials. For example:

- delamination/debonding of FRP elements
- decay/insect attack of timber/LVL elements
- corrosion of steel components
- corrosion/tightness of fixings.

In addition, cracking of components, distortion/displacement, crushing, excessive deflection along with impact and fire damage are all indicators of deterioration/distress.

3 Common defects observed in existing structures

3.1 Concrete bridges

The following section lists the various types of reinforced and prestressed concrete bridges and generally lists the main problems associated with each type.

3.1.1 Monolithic and simply-supported T-beams

Most monolithic structures are T-beam bridges with the whole structure cast insitu. Spans tend to be small but groups of as many as five continuous spans may be built this way in a bridge. This puts strains on the columns of the piers and at the abutments due to temperature movements, and it is not uncommon to see a crack and signs of movement around the beam/wall joint at the abutment. There may also be signs of tension cracking in the face of the columns of the furthest pier from the centre of the span group, due to movements and temperature. These structures are often overstressed in negative moment with cracking and staining observed at the underside of deck at the beam/deck/pier diaphragm joints.

The T-beam bridges often have sufficient shear reinforcement near the supports and diagonal shear cracking may be observed as far away as one third of the span from the support. The abutments and wings were usually cast as one and heavy cracking, spalling and movements may be observed at the wing joint especially where high abutment walls were built.

The simply-supported precast T-beam structures tended to be a later design with improved shear reinforcement of the beams and hence shear cracking is not normally seen. Some flexural cracking of the beams will normally be seen at midspan especially on structures which carry a reasonable number of heavy loads. Some beams had a locating dowel at one end of span which made that end of beam fixed with the other end free to move. The allowance for movement was often lost, with the consequence that the beam moved relative to the dowel, cracking and sometimes spalling the ends of the beams. The support directly under the beams also tended to spall due to friction, as a layer of malthoid was all that often separated the beam and substructure.

3.1.2 Precast 'I' beams

Precast 'I' beam construction began in the early 1960's, using precast high strength prestressed concrete beams with spans up to 22 metres approximately. These beams have generally performed well over the years.

The National Association of Australian State Road Authorities (NAASRA) beam sections came into use in 1970 and Type 3 and Type 4 girders have been used extensively for spans up to 25 m and 31 m respectively. Longer spans have been accomplished by casting load bearing diaphragms at the piers which encased the ends of the beams to create continuous spans. The beams were also connected on the bottom flange by heavy steel bars welded together. In recent years a 'bulb tee' section has been used in place of the Type 4 NAASRA beam for spans up to 36.5 metres.

The biggest problem associated with prestressed beams for large spans is the amount of hog of the beam, especially as they continue to hog further after delivery until loaded by the weight of the bridge deck. The beams can also crack towards the ends due to stressing if insufficient end steel in provided.

3.1.3 Precast prestressed inverted 'T' beams

These beams were used during the 1970's to produce a flat soffits to bridges crossing the highways. This was done for aesthetic reasons as the flat soffit is more appealing to the driver than the interrupted underside of an 'I' beam bridge. Spans were usually in the region of 10 metres. These beams were not an efficient section and lost favour with designers. No problems have been encountered with these types of structures to date. Top slab construction or concrete infill between beams have both been used.

3.1.4 Box girder bridges

Box girder bridges have been used extensively on or over freeways in Queensland. They are generally cast-in-place and then post-tensioned. Some box girders have been precast in segments and post tensioned when erected in place. Problems can regularly occur during construction and at post-tensioning.

The major maintenance concern for these bridges is where grouting around the post tensioning is incomplete and does not adequately protect the steel tendons. Serious concerns have been identified in some overseas countries where de-icing salts are used on the deck but to date no evidence of tendon corrosion has been observed in Queensland bridges.

3.1.5 Prestressed voided flat slab bridges

A number of cast-in-place prestressed voided flat slab bridges have been built on or over freeways and highways and these provide an attractive shallow depth superstructure, ideal for very wide bridges and with spans to approximately 34 metres. Problems with flotation and distortion of the void formers have been experienced during construction, but these structures are relatively cheap, aesthetically pleasing, and have performed well up to now.

3.1.6 Reinforced concrete flat slabs

These structures are monolithic cast-in-place and have performed very well with the slab providing considerable lateral load distribution. Structures can be continuous over a number of spans, hence there is a possibility of cracking of the columns due to movement.

The slabs themselves often have a shrinkage crack which runs almost directly down the centreline of the slab. Provided this remains dry it is of no concern.

3.1.7 Precast prestressed deck units

Introduced in 1954 these units are held together by transverse tensioning rods in cored holes.

This has been the principal form of bridge superstructure constructed over the last thirty years. These 596 mm wide, rectangular section, voided planks cover the span range from 8.0 to 27.0 m varying in depth from 300 to 900 mm respectively.

Typically these elements are erected with a 20 mm gap between adjacent units which is subsequently filled with poured mortar. The mortar acts both as a shear key and a means of providing an even bearing surface between units for the transverse prestressing forces. The latter is applied by way of transverse stressing bars slotted through cored holes in the units. Following the application of prestress force the gaps around the bars and joints at the ends of units, at piers and abutments, are also filled with mortar.

The mortar in the joints inevitably cracks as a consequence of shrinkage and girder deflections and rotations. This permits water to penetrate from the surface to the unit soffits and substructure elements. Recently there have been failures of the transverse stressing bars which have corroded as a consequence of this. Additionally, in regions where Alkali-Silica Reaction (ASR) is a problem that reaction is exacerbated by water leaking through the deck. The extent and severity of cracking and the production of reaction products are more pronounced in the wetter areas of the bridge. That is, adjacent to the joints between units and spans and around the kerb unit. It is imperative that deck drainage is efficient on those structures and that any cracking of the surfacing around deck joints is sealed. Current designs detail waterproofing of the longitudinal joints between units to avoid the problems discussed above.

The anchor plate and ends of transverse stressing bars are usually exposed. In an aggressive environment, these components may be heavily corroded. In addition, the threaded ends of the transverse stressing bars may not have sufficient length. In some instances these components are installed in the formed voids on the external concrete plank which are filled with mortar to protect them from corrosion.

Generally, the deck units alone comprise the superstructure however a reinforced concrete deck slab acting compositely with the units is often adopted in lieu of the transverse prestressing. Currently the slab is made continuous at fixed pier joints to improve ride and minimise the number of pier joints.

Erection of deck units on elastomeric bearings, especially at expansion joints, can be compromised by excessively hogged units or lateral shearing on headstocks with crossfall unless the unit is supported and braced adequately until the levelling layer of epoxy has cured.

3.1.8 Precast prestressed voided 'T' beams

These standard beams were originally developed in 1986 by VicRoads and subsequently introduced in Queensland. With initial spans from 8 to 19 m, the original design has evolved to include Super-T and T-roff beams with span ranges up to 35 m and beam depths up to 1.8 m.

The potential problems identified with these beams include high neoprene bearings placed on sloping headstocks beneath the T-beams and loss of cover due to void formers floating during fabrication.

3.1.9 Decks and overlays

Reinforced concrete decks are usually cast-in-place over beams. For high structures or bridges over highways and railway lines, thin precast prestressed concrete formwork slabs or sacrificial formwork is usually used to negate the need for stripping after casting the deck.

The concrete beams have ligature bars which project into the deck for composite action, whilst steel beams and girders have welded stud or other anchors at their top to provide composite action. On some older bridges a bevelled concrete cap was cast between the deck and beams. Cracking of the cap can occur along the fillet line at the deck, or cracking coinciding with the location of the shear connectors may be visible. Unless severe, this cracking is not cause for concern.

A 50 mm or larger thickness of bituminous wearing surface is generally laid over concrete decks however earlier concrete decks were increased in depth by 12 mm to provide an unsealed wearing surface. This practice has now been discontinued due to temperature cracking of the surface which allowed moisture to penetrate into the deck concrete.

3.1.10 Diaphragms

At the ends of the deck a stiffening beam will be noticed joining the ends of the beams. This diaphragm (cross-girder) may be the full depth of the beams, but on some structures it will only be in the order of 200 to 250 mm in depth.

Diaphragms may also be found at midspan or at the third points to provide web stiffening against debris loads and impact forces and aid in load distribution between beams.

On precast prestressed 'I' beam bridges continuous for live load, a wide heavily reinforced load bearing diaphragm can be found at the piers. This diaphragm is required to support the full superstructure loads and transfer that load back to a pier or to isolated columns which form the pier.

All these diaphragms should be checked for cracking.

3.1.11 Kerbs, footways, posts and railing

Most early concrete bridges used either narrow kerbs (sometimes tapered in cross section) or wider kerbs tapered (in plan) at the ends. These kerbs had a barrier facing which was stepped back from the kerb face. This caused a dangerous situation whereby errant vehicles could ‘take-off’ and land on top of the barrier rather than be redirected by it.

Where footways are constructed on bridges they should be inspected for pedestrian safety, i.e. ensure level of precast or cast-in-place footway slabs is good with no depressions or rises which could trip pedestrians. Moisture will penetrate the footway slabs and adequate drainage of the area under the footway is required. If drainage is not adequate weed growth will form and the underside of deck will form efflorescence with the dampness penetrating the deck.

Many different forms of post and railing have been used on concrete bridges ranging from guideposts, timber posts and rails, reinforced concrete posts with precast reinforced concrete rails, reinforced concrete posts with steel tube rails, steel channel posts with steel guardrail, rectangular rolled hollow steel posts and rails, and reinforced concrete new jersey barriers with steel posts and rail on top.

Pedestrian grating is usually associated with footways, or on pedestrian bridges, and should be inspected for damage and tightness of the attachment bolts.

For all bridges it is important for the steel guardrail on the approaches to attach to the bridge endposts or to continue over the bridge. This will prevent the possibility of a vehicle hitting the approach rail and being redirected directly into the endposts or striking an unprotected endpost.

3.1.12 Abutments

Abutment types vary but will generally be one of the following types:

- spill through abutments using a reinforced concrete headstock supported on driven precast concrete piles or of a frame type with reinforced concrete columns supported by a footing below ground
- wall type abutments either reinforced or mass concrete
- wall type consisting of straight columns and a headstock with infill wall panels between the columns
- masonry walls
- spread footing
- sill beams behind a reinforced earth wall.

Spill through abutments are possibly the most common type to be found and usually have little or no cracking of the headstock, except for shrinkage cracks. Frame type headstocks are more highly stressed and some flexural cracking may be found at midspan between the columns, or over the columns. Loss of retaining fill in front, beneath and behind the headstocks is also a common problem which requires correcting to retain the embankment fill behind the abutment.

The columns or piles are not usually a problem although cracking of the front face of piles has been noticed where the superstructure has propped the abutment against large movements of the embankment fill. This is only a problem if the cracking becomes severe. The ballast walls will often crack if beams bear hard against them or if an overhanging deck puts pressure of the top of the wall. This cracking is not considered very important provided excess moisture is not allowed through the walls.

Wall abutments are usually in good condition with differential movement between panels the only area of concern. Mass concrete walls are usually small in height and have only movement problems or in some instances scour problems of fill in front of, and beneath, the footing. Wall abutments consisting of columns with headstocks and thin infill panels can have cracking from the effects of earth pressure and shrinkage.

The side wings on the high abutment walls often move relative to the abutment walls due to earth pressure. The wings are not normally self-supporting and rely on a concrete key or few bars of light reinforcement to hold them in place. Cracking and differential movement between the wing and the abutment wall are quite common and can be a problem if severe.

Highway and freeway structures are designed to have reinforced concrete approach slabs which rest on top of the ballast walls. These are installed to eliminate live load earth pressures behind the abutments and to provide a smooth transition onto and off the bridge for fast moving and heavy traffic, thus reducing the impact loads on the structure.

Stone masonry abutment walls have been constructed on older structures. Care should be taken in assessing these walls for possible signs of settlement of the blocks, settlement cracking or cracking of the wall especially under heavily loaded areas. Where loadings on the wall are at isolated points such as girders rather than a distributed load, a reinforced concrete cap may be cast on top of the wall to distribute the stress.

If this cap overhangs the masonry for a bridge widening, particular attention should be noted of the edge loading occurring on the masonry.

3.1.13 Piers

Piers of various types include headstocks supported on piles or columns, wall type piers some of which consist of columns with a headstock and thin in full panels, straight walls of constant or variable thickness, box type concrete piers and masonry piers.

Cracking of these pier types will be similar to the cracking mentioned above for the abutments. With the higher wall piers horizontal cracking may occur around the construction joints.

With continuous superstructures and large movements occurring at the abutments, horizontal cracking of the pier wall or column face can occur as bending pressure is exerted on the wall.

Bending pressure can also be put on high slender columns or piles if the bridge is on a large skew or a sharp circular curve, causing lateral cracking of the piers low down.

Long monolithic T-beam bridges often have split piers at the deck expansion joints. Cracking and spalling is a problem with these piers due to the high moments on the slender sections.

Portal frame and cantilevered headstocks have, in some instances, been found to have theoretically insufficient bending or shear capacity. This could lead to the development of structural cracking in the headstock.

Many of the older structures have poor quality sandy concrete which can suffer severely from the action of water, sand, pebbles and grit as they wash past. This can significantly reduce the amount of cover concrete to the steel reinforcement and guniting may have been used to reinstate the concrete surface.

3.2 Steel bridges

Composite steel beams with reinforced concrete decks were used in the past for longer span structures but are seldom used today. The reasons for this were cost, (fabricated plate girders are much more expensive than prestressed concrete beams) and future maintenance problems with repainting.

These superstructures also tend to deflect substantially and continuous steel girders vibrate with loading of adjacent spans. Because of this movement under load, the reinforced concrete deck will often crack through at approximately the third points of the spans. Moisture, corrosion and efflorescence at the cracks will normally be seen on these type of structures.

Steel beams should be checked for signs of corrosion and the condition of the paintwork noted. Simply supported beams should have steel angle cross-frames or concrete diaphragms at midspan to prevent lateral buckling and aid in stiffening the beams. Continuous beams should have these at the supports and at midspan. Splice plates on the web, top and bottom flanges should be inspected to ensure no weld cracking or separation has occurred. All welded connections, splices and stiffeners should be closely inspected for any signs of cracking of the weld or metal immediately adjacent to it.

Bolted and riveted connections require inspection to check whether all connections are tight, intact and the protective cover is in good order. Loose bolting can sometimes be detected by cracks in the coating system, movement of the bracing or by associated noises as transient loads cross the deck.

Any signs of excessive wear at pinned joints in trusses or other movement joints should be observed and recorded.

Areas around the junction of members should be inspected for straightness as these can be the first sign of permanent deformation resulting from buckling of compression flange or member or a sign of inadequate bracing.

The thin steel sections are also susceptible to permanent deformation caused by vehicle impact and if severe can significantly reduce the load carrying capacity of the structure. This can be caused by impact from a high vehicle travelling under bridge damaging the bottom flange or chord member, or by vehicles at deck level causing damage to through girders and trusses.

3.3 Timber bridges

The following section is a general description of common defects found in timber bridges. For a detailed description of element-specific defects, refer to Parts One and Two of the *Timber Bridge Maintenance Manual*.

3.3.1 Timber girders

Timber girders may be either round, hewn or sawn. Hewn or sawn girders will generally not have any outer sapwood except in the case where CCA preservative treatment has been applied.

Timber girders should be inspected for pipe or external rot at their maximum stress location at midspan. Inspection at the girder ends should also be carried out as pipe rotting is generally more severe at these locations. Girder ends are prone to crushing failure when excessive loss of section has occurred.

The girders should also be checked at their ends for splitting (some timber girders may have anti-split bolts at their ends to control any splitting), they should have full bearing on corbels and they should be checked for end rot especially at abutments where moisture or wet fill is prevalent.

Splitting of timber girders can affect their performance and working life considerably. Much of the splitting will be along the grain and, unless severe, is not of significance unless it allows considerable moisture into the splits. Spiking of the decking to the timber girders can cause splitting at the top, and, with the presence of moisture and vibration of the spikes under traffic, spike rot of the girders occurs. For this reason spiking of decking directly to girders should be avoided. Generally, decking will be spiked to a sacrificial spiking plank on the outer girders with no spike connection to inner girders. Longitudinal cracking of girder ends, when combined with large pipe size, will lead to the girder section being split into a number of discrete segments which will reduce shear and crushing strength at the support ends.

If the girder is severely split in the vertical plane loading can tend to widen the splits causing premature failure. By far the most dangerous splits are the fracture of the timber due to overloading, and the split which starts from the bearing area and travels diagonally across the timber grains towards the top of the girder. In both cases the girders will require relieving or replacing, though steel banding could control the diagonal splitting if load limits are placed on the structure.

Other problems which may occur with timber girders are the presence of rotting knot holes (especially if at midspan), sagging of the girder at midspan, or excessive deflection of the girder under live load due to poor lateral distribution of the decking, or the member being too small for the span.

Loss of section due to termite attack can seriously affect the performance of timber girders and care should be taken in searching for any evidence of their presence.

3.3.2 Corbels

Corbels should be checked for splitting and pipe rot at their ends. If piping or splitting is severe then crushing of the corbel can occur with subsequent excessive vertical movement of the timber girder at the end. Many corbels have anti-splitting bolts through their ends in an attempt to prevent crushing from occurring.

3.3.3 Decking (timber and steel trough)

3.3.3.1 Transverse planks

The most common form of decking consists of transverse planks spiked to the outer girder spiking plank with no mechanical connection to the internal girders. The ends are also restrained by the bolting down of the kerbs and jacking of internal girders (cambering) is used to provide tightness to the whole superstructure. However, this type of structure will always work loose due to shrinkage and creep in the members and will require continual tightening of bolts and recambering. Timber running planks are often used with transverse deck planks and these planks aid in load distribution to the deck planks. The running planks are usually of a thin section (about 50 mm thick) and being usually spiked down, tend to work loose quite easily. They tend to split quite easily, requiring constant replacement and form a moisture trap which hastens rot of the decking beneath. Longitudinal timber distributor planks are often bolted to the bottom of the decking to reduce differential movement between the transverse deck planks under the action of wheel loads. Though distributors may help with load support in deteriorated decking, they are of no benefit in improving the distributing of loads to the girders. Most bridges have an asphalt or penetration macadam wearing surface over the transverse decking, but the surface becomes quite bumpy and cracked due to movement of the decking below, although it does offer improved load distribution. It also tends to build up a reservoir of moisture which rots out the timber at a quicker rate.

Transverse deck planks should be inspected for end and top rot (particularly in the kerb region) bulging on top due to ingress of water, sagging at midspan due to excessive span length, fracture and severe splitting. Severe splitting and top rot can often be caused by spiking of decking and the practice should be discouraged except at the outer edge connection.

The inspector must always be alert for signs of termite damage as the consequence on these small sections can be severe.

3.3.3.2 Plywood decking

CCA treated plywood is often used for deck replacement on timber bridges.

Differential movement of sheets under traffic loads and inadequate sealing of joints can cause damage to the roadway surface. As well, the long term performance of the ply in wet tropical areas, or where submergence is common should be monitored to check for delamination of the plys. The exposed outer ends of the sheets should also be examined for evidence of delamination.

Common defects of plywood decking and the root causes include the following:

- Loss of cross sectional area in the bottom layer of due to abrasion between the girder and plywood panel.
- Abrasion between sheet joints caused by material from the deteriorated deck wearing surface working their way between the sheets.
- Deck wearing surface had been lost over significant areas resulting in the top scarf jointed layer being worn away due to traffic abrasion in places (refer Figure 3.3.3.2a).
- Deck bolt washers are less than the specified size causing localised crushing of wood fibre around the bolts. This has contributed to loose panels which are able to ‘grind’ and flog on the girders.
- Broken pieces of deck wearing surface work their way down between the sheets contributing to the grinding action of plywood sheets on the girder seating.
- Plywood panels are loose, evidenced by the pronounced transverse cracking in the deck wearing surface over every panel joint and by the total loss of the bottom outer layer of the plywood - girder interface.
- The underside of the bridge has been subjected to severe fire damage resulting in the loss of the bottom layer and spongy material above the fire damaged section. The latter could be indicative of changes to the wood structure due to the heat of the fire.

Figure 3.3.3.2b illustrates failure of a plywood panel within the wheel track.

Figure 3.3.3.2a – Abrasion of wearing surface

Figure 3.3.3.2b – Failure of plywood deck panel

3.3.3.3 Longitudinal decking

A relatively uncommon form of timber decking consisting of transoms and longitudinal decking has been used in the past.

The transoms should extend across a minimum of three beams unless designed especially for simple spans. They should be firmly bolted to the beams and all bolts should be regularly checked.

Longitudinal decking should be laid in long lengths and should be securely bolted to the cross beams at their ends and at alternate intermediate transoms. This is done to stop flexing of the longitudinal decking under load, and reducing the pulling motion that shears the bolts through the ends of the longitudinal decking planks, which is a commonly occurring problem.

Longitudinal decking should be laid with the heartwood down to prevent it rotting and splitting at the centre, or possibly curling up at the edges.

As the timber shrinks and dries out gaps will form between the planks and jacking of the deck may be required to close up the gaps with insertions of thin sections. This is especially important on bridges used by cyclists, and timber bridges should be signed to warn cyclists of the possible dangers when crossing the bridge.

3.3.3.4 Steel trough decking

Many timber bridges now have steel trough decking replacing the timber decking. The troughs were initially filled with premix asphalt or mass concrete to a level of approximately 50 mm above the top of the trough sections. Neither of these infills has performed well as both are porous, permitting water entry to the trough decking. Compaction of bituminous infill under traffic loads occurs and approximately two to five years after opening (depending on traffic volumes and loads), the infill should be resurfaced to regain both longitudinal grade and lateral crossfalls. It is vital with this type of decking to maintain a crack free surface with good drainage to remove all surface water from the deck, so that it will not seep through the infill and lay in the steel trough causing corrosion to occur. Some trough sections were tack welded along their joints whilst others have been bolted or screwed together. A check should be made of the joining arrangements in case the trough sections are tending to spread under load. If this problem is occurring, it will normally be reflected in the road surface above as an area of heaving, dips or even pot holes in the infill or areas of heavy cracking. They are signs that the trough sections are deflecting excessively under load or are not being effectively held down to the girders. Refer to Figure 3.3.3.4a and Figure 3.3.3.4b for commonly visible defects.

Concrete infill with mesh reinforcing over the trough decking has been the most successful infill material used. In addition, where trough decking is in very poor condition, a number of bridges have had a structurally reinforced deck poured, using the trough profile as permanent formwork.

Figure 3.3.3.4a – Corrosion of joints between trough sections

Figure 3.3.3.4b – Cracking and perforation of steel trough decking

3.3.4 Kerbs, posts and railing

Visual inspection should be conducted of the kerb condition and bolting to the decking or beams. The kerbs should be firmly held in place as the barrier posts rely on this for strength of support.

The endposts may be round timber and suffer from settlement, splitting, sap rot, base rot, piping, and top rot due to weathering. If the post can be moved by hand it is usually a sign that replacement is required, though in some cases this can be caused simply by a lack of embedment in the fill.

Inspection usually consists of visually inspecting the bolting, paintwork and damage caused by glancing blows from vehicles.

Standard timber rails are mainly used on timber bridges but steel guardrail is also reasonably common. Connections need to be inspected for rigidity, and paintwork inspected for traffic safety reasons. With timber rails, rot and splitting may require early replacement of some sections.

3.3.5 Piles

Piles can be classified into two main groups; those which take vertical loads and support headstocks, and those which take moments such as wingwall piles or stream fender piles. Abutment piles are required to take both vertical loads and horizontal earth pressure loads.

Areas where rot is most likely to occur are at ground level, normal water level (usually 300 to 600 mm below waling's) or around areas of numerous bolt holes such as waling's and crossbracing.

Piles which take moments are particularly susceptible at ground or normal water level where maximum stress and suspected rot areas coincide. If pipe rot has been detected in these critical areas the extent of decay needs to be defined so the length of repair or replacement of the pile can be determined (refer Figure 3.3.5a).

Care also needs to be taken in determining natural ground level as scour, filling or siltation may have occurred. If filling or siltation has occurred, the pile may have substantial pipe rot well below the current ground level. If the pile has rotted out below ground and moving under load, a void will be seen around the pile. Under load the pile will be seen to visually move. If this occurs in water, ripples will be seen to emanate from the moving pile. In scoured areas the pile will need to be inspected higher up at what was originally the ground level.

Piles should be visually checked for areas of rot or splitting in the loaded areas at the top, especially splits originating beneath the headstocks (refer Figure 3.3.5b).

Where the bridge is submersible, the adequacy of the headstock/pile bolted connection should be checked. Weakening of the piles above this section due to pile rot may allow the superstructure to float off. The presence of the top pile strap bolt should also be confirmed.

Termites are a continual problems with timber piles in all areas of the state. The termites can enter the piles as low as 300 mm below ground, but usually enter via splits in the timber. Their presence can be seen with dirt galleries in the splits or along the outside of the pile. They may also be encountered stuck to the probe when testing the pile for rot. The termites eat out runways within the timber and when probing the test hole it feels as if you are scraping over a lot of thin timber sections.

Piles can often wear away at ground level or at bed level due to the action of abrasive gravels or sands and this should be checked. The abrasive gravels occur in the mountainous regions and the wear can usually be seen, but abrasion by sands usually occurs at the mouth of the rivers and is due to sand movement with the tides. Structures in these locations should have the pile diameters at bed level checked by divers to ascertain the loss of section.

Figure 3.3.5a – Decay of pile below ground level

Figure 3.3.5b – Splitting of pile beneath headstock

Timber piles in marine situations can also suffer attack from teredo. This attack can occur anywhere between bed level and mean low tide level. Presence of teredo can be judged from either sacrificial Oregon timber attached to the pile group, or by smooth runways along the hardwood timber in the mean low tide area (they may often only attack the softwood) or by small 5 to 10 mm diameter holes in the piles below water. Teredo will consume the interior of timber piles with damage going completely unnoticed until failure of the pile below water occurs, hence the importance of early detection.

3.3.6 Waling's and crossbraces

Waling's and cross bracing should be visually checked to ensure that the members are adequately stiffening the piles and providing a rigid frame against the action of the stream and possible debris and log impact forces. Waling's are usually encountered 300 to 600 mm above normal water level and give a good guide as to the relative water level at the time of inspection, i.e. if the water level is too high then the timber piles should be reinspected when the level drops to normal water level. Waling's can also be good guide as to whether scour or silting is occurring at the pier.

3.3.7 Headstocks

Most headstocks on timber bridges consist of sawn timber, approximately 300 mm x 180 mm in a section. Headstocks consisting of solid hewn sections also exist, but are much less common.

Inspection of headstocks should include the following areas:

- check for presence of termites
- check for top rot due to the presence of wet fill
- check for weathering or end rot
- check for splitting
- check for any rot or separation of headstocks that are spliced at an inner pile
- if the beams are not directly over the piles then the headstocks should be checked for sagging, indicating they are being overloaded
- check for any settlement of piles causing a sag in the headstocks
- be especially wary of loaded timber overhangs
- check that the headstocks have mechanical support on the piles and are not purely relying on bolting to transfer their loads (headstock seating may have been removed to allow placement of pile bracing)
- check all bolting is tight and in place. Severely corroded bolts are to be replaced
- check for loss of section due to excessive cuts in headstock (typically in the vicinity of the bracing).

3.3.8 Abutments and piers

3.3.8.1 Bed logs and props

Some timber bridges have bed-logs stacked on top of each other to form abutments, whilst others have props resting on a bed-log to form a relieving abutment in front of the original abutment.

Items to check include:

- pipe rot in main load bearing areas

- load bearing of the timber girders or props on the bed-logs
- check for severe crushing of the bed-logs under loaded areas
- check for excessive splitting or end rot of the bed-logs
- check for leaning of the bed-logs.

Sometimes a timber bed-log may be placed in front of the other bed logs to support the fill on which the bed-logs bear. These bed-logs do not support the girders but are still important in retaining the fill and preventing scour beneath the bearing bed-logs.

Props are used to transfer vertical load from suspect piles or suspect abutments and usually bear on bed-logs or heavy sawn timbers. The props should be inspected for rot if they consist of round or hewn timber which still has the heartwood within. If the prop is of sawn timber there can be no pipe rot, but its condition such as end bearing support, connection to bed-log, splitting etc. should be noted.

The prop must be securely attached to the girder or relieving headstock, and capable of taking the direct load. Stability of the props is also important and any leaning prop must be listed for repair.

3.3.8.2 Headstock on piles

The majority of timber abutments and piers will be of this form. Refer to paragraphs 3.3.5 and 3.3.7.

3.3.8.3 Abutment sheeting, ballast boards and wing sheeting

The abutment and wing sheeting and ballast boards are structural elements. Abutment and wing sheeting may consist of timber planks or precast reinforced concrete units, placed behind the piles to hold the embankment fill in place. The inspector should check for rot, cracking, bulging and undermining by the stream.

Ballast boards can consist of timber sheeting or precast reinforced concrete units. The inspector should again look for rot, cracking, bulging or breaking out of concrete. Once again, the function of the member is to adequately retain the embankment fill, and, provided the rot, cracking or bulging is not too extreme, these members are usually adequate.

3.4 Other structure types

3.4.1 Box culverts

The early type box culverts were cast-in-place and many suffer from cracking and spalling due to lack of concrete cover or ingress of moisture. Once repairs are required to these structures they tend to be ongoing problems as the other areas fail due to general dampness through the porous concrete.

A common form of construction in the 1950’s – 1960’s comprised in situ walls supporting reinforced concrete slabs dowelled into the piers. Failure of these dowels resulting in movement of the deck slab under load has been observed in a number of these structures.

Precast concrete crown units have been used extensively and have generally been found to perform satisfactorily. However, in many structures fabricated in the 1960's, where calcium chloride was used as an accelerator, the reinforcement has corroded severely leading to extensive spalling of the cover concrete.

Link slabs have been used on multi cell culverts to reduce construction costs and time. The link slabs take the place of the intermediate row of precast crown units by spanning across the gap between alternate rows of crown units. The link slabs may be either precast in a casting yard, or cast on top of the culvert base slab and simply lifted into position as required. No service problems have been noted with these slabs at present.

Proprietary modular culvert systems have also been adopted on the network. These comprise discrete wall and roof sections that are designed to be connected through a combination of dowels and a series of fabricated bolted connections.

Construction tolerances have proven to be a problem and the jointing system should be inspected for completeness, fit and tightness of bolts. In addition, the panel alignment has also been compromised in some areas and the structure should be checked for consequential damage such as cracking or spalling caused by bolts or panels bearing excessively on the panel faces.

3.4.2 Pipe culverts

Early pipe culverts were predominantly of masonry construction, formed from engineering ‘red’ bricks or similar materials. Where conditions were found to be suitable, the pipe was carved through solid rock.

The majority of these structures were built around the early 1900’s, and the ones inspected to date appear to be performing satisfactorily, with no major defects found. However, a number of minor defects have been identified, such as perished mortar, groundwater infiltration (resulting in limescale leaching and the passing of fines through the culvert lining) and spalled brickwork, which are attributable to general deterioration over the life of the structure.

Modern pipe culverts are typically constructed from precast concrete segments or corrugated steel sections, and may be circular or elliptical in shape.

Pipe culverts should be inspected for the presence of corrosion (metal culverts and reinforcement), cracks, spalls, line, level and stability of headwalls and wingwalls.

3.4.2.1 Precast reinforced concrete culverts

Precast concrete pipe culverts have been used in Queensland for over 50 years. Currently there are some 25,000 concrete pipe structures recorded in the Structures Information System (approx. 150 major culverts, remainder are minor culverts).

Potential defects of most concern relate to cracking of the pipes. Cracking in pipes can compromise the strength, durability and function of the pipe over the design life. Cracked concrete allows water/air through the cracks and will trigger or speed up deterioration processes such as carbonation of the protective concrete cover and actual corrosion of the reinforcing steel. Once the steel starts to corrode, spalling of the surrounding concrete occurs resulting in a weakening and eventually collapsing pipe. Depending on the size of pipe, major defects can cause loss of the road function and can pose significant safety hazards to road users.

Cracking may be initiated by any number of reasons such as:

- incorrect design
- inadequacies in manufacture
- incorrect handling/stacking or transportation
- incorrect installation
- overloading during construction.

Figure 3.4.2.1a and Figure 3.4.2.1b illustrate examples of cracking and spalling in precast concrete pipe culverts.

Figure 3.4.2.1a – Cracking of pipe walls

Figure 3.4.2.1b – Loss of cover/spalling concrete in pipe crown

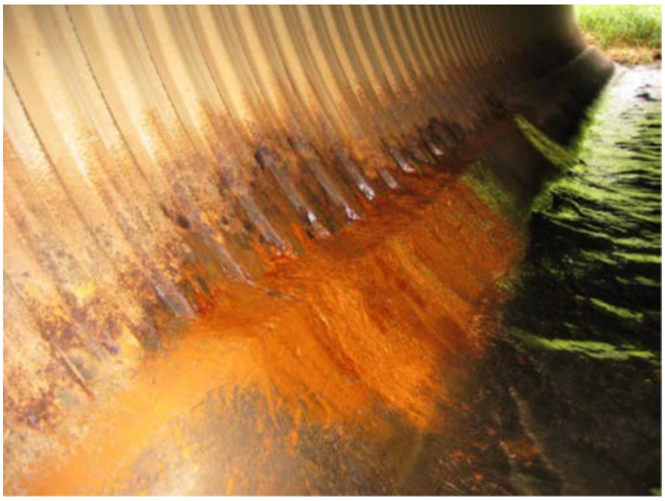
3.4.2.2 Buried corrugated metal culverts

Buried corrugated metal (BCM) culvert structures are recognised worldwide as a high risk structure, comprising a thin wall section of steel or aluminium that is prone to corrode longitudinally at the top of the wetted area or in the invert. If left unattended the corrosion can lead to perforation of the corrugations. Figure 3.4.2.2a and Figure 3.4.2.2b illustrate the typical location of corrosion while Figure 3.4.2.2c and Figure 3.4.2.2d show examples of perforated culverts.

Aluminium culverts are known to have greater corrosion resistance than steel culverts but performance is sensitive to surrounding soil conditions. Aluminium culverts are more susceptible to abrasion effects than steel culverts unless suitable protection is provided.

Figure 3.4.2.2a – Typical location of wall corrosion in BCM culvert

Figure 3.4.2.2b – BCM culvert wall corrosion above invert protection



Once a metal pipe has perforated, two failure modes are possible:

- In dry conditions, the soil pressure eventually causes the culvert to shear through and the tube folds in on itself. The soil above arches and supports the embankment and traffic loads but gradually settles, creating a dip in the road surface. This gradually increases until it becomes dangerous, particularly to small vehicles (such as motorbikes) and high speed traffic.
- Heavy rain and associated heavy stream flows in a corroded culvert can rapidly erode the compacted soil embankment causing voids around the culvert and eventual piping failure. Where the embankment height is greater than the pipe diameter, soil arching is maintained until it is destabilised by the formation of a void in the embankment, resulting in the sudden collapse of the road and the consequential risk of vehicles dropping into the open barrel of a fast flowing stream.

Figure 3.4.2.2c – Perforation of BCM culvert invert

Figure 3.4.2.2d – Perforation of BCM culvert wall

3.4.3 Large traffic management signs

Large cantilever signs, butterfly signs and sign gantries are increasingly prevalent on the network and constitute major structures in their own right.

Sign and signal gantries either span or cantilever over part of the carriageway.

The consequences of any failure or partial failure of one of these structures has the potential to cause significant disruption to the network and potential loss of life.

High wind forces on signs and other attachments will produce large twisting forces on supports, connections, columns and base connections. Furthermore they are also susceptible to wind induced vibrations with associated potential fatigue related defects (particularly in hold-down bolts and welded connections) which could lead to collapse of the structure onto trafficked lanes.

Key areas for inspection include:

- missing, loose or damaged nuts/bolts
- cracked welds
- butt welds at structural connections
- corrosion

- splits or ruptures in columns and stiffeners
- impact damage
- tilting columns
- crushed/missing mortar beneath base plates
- exposed levelling nuts beneath base plate (levelling nuts should not be engaged in carrying load after mortar has been placed)
- base plates or other steelwork which has been incorrectly (i.e. not in accordance with the drawings) encased in grout or concrete (any affected components will need to be exposed to determine the extent of any corrosion)
- cracking/spalling of concrete around base plates.

Nuts on hold-down bolts should have been checked at installation for tension and marked. The inspector should check the marks and note if there has been any movement or loosening of the nuts.

3.4.4 Retaining walls

Retaining walls are any structure where the dominant function is to act as a retaining structure for embankments or fill slopes be they above, below or either side of the carriageway.

A variety of structural forms are employed across the network including:

- Gravity wall – resist earth pressures through own self weight. Examples of gravity walls include:
  - mass concrete monolithic walls
  - unreinforced masonry walls
  - gabion baskets (i.e. woven steel wire baskets filled with stone)
  - crib walls (reinforced concrete or timber crib units filled with free draining material)
  - reinforced soil/mechanically stabilised earth walls (soil nailing or anchoring using steel or geotextile reinforcement to stabilise retained material).
- Cantilever on foundation wall – comprise a vertical wall rigidly fixed to a horizontal foundation slab. Horizontal earth pressures are transferred to the foundation (primarily in bending). These types of wall are typically constructed of reinforced concrete.
- Embedded retaining wall – these types of wall are similar to cantilever on foundation walls with the exception that there is no horizontal foundation. Retention of fill is achieved through depth of embedment. Examples of embedded retaining walls include:
  - sheet piles. driven steel, concrete or timber piles
  - insitu concrete bored pile walls. Can be contiguous or secant piled walls.
- Diaphragm walls – insitu or precast reinforced concrete walls placed into a narrow trench stabilised with bentonite slurry. The bentonite slurry is displaced during construction.
- Soldier pile walls – comprise driven (steel, timber or precast concrete) or insitu concrete vertical piles installed at regular centres with sheeting spanning between the piles. Sheeting may be steel, precast concrete or timber.

Defects/deterioration associated with the various wall types described above can be attributed to the construction material (s) or stability of the retained (or founding) material.

Material related defects will typically be as described in Section 2. Any connections/fixings utilised in the wall construction type must also be considered, with particular attention paid to tightness, corrosion and missing or incorrectly installed fixings. Embedded materials not accessible for inspection (e.g. soil nails, anchors, etc.) may require specialist inspection to evaluate condition unless consideration was given at time of design/construction (e.g. inclusion of additional ‘sacrificial’ nails able to be removed for inspection or installation of load cells to monitor tension in embedded components).

Indicators of potential issues with stability of retained or founding material can include:

- cracking/slumping of carriageway or shoulder parallel to the retained face may indicate settlement, outward movement or loss of retained material
- change in alignment of the top face of the retaining wall or guardrail/barrier
- change in angle of the retaining wall indicating rotation of wall
- erosion or removal of material to front face of the retaining wall
- blocked/inadequate drainage (weep) holes to relieve pore pressure behind walls.

In addition to the above, the impact of any proposed change in land use adjacent to a retaining structure (e.g. widening or realignment of carriageway (even temporarily to facilitate maintenance) must be carefully considered as the effects may not have been considered at the design stage.

3.5 Causes of deterioration not related to construction materials

A number of components need to be inspected which are not related to defects in construction materials used in a structure but which, if not observed or maintained, could be a cause of future deterioration.

3.5.1.1 Deck joints

Various types of expansion joints have been used in the past to cater for movements of bridge superstructures. Early bridges had small simply supported spans and hence only small movements needed to be catered for.

These joints included materials with small compressions such as cork, bituminous impregnated fibreboard, butyl impregnated polyurethane foam, styrene and foam strips. Asphalt, rubberised bitumen or polyurethane were often poured over the top in an effort to seal the joint from moisture penetration. Many of these joints failed to seal due to the joint material debonding or being inelastic. If the sealant was placed too high in the groove, traffic tended to crack the sealant and rip it out.

For small expansion joints a repair being used at present is to pour a polymer modified bitumen (Mobil N345 or ‘megaprene’) into the joint with a thickness of approximately 20 mm. Care must be taken to ensure overfilling does not occur and a 6 to 10 mm depth from the top is required so that traffic will not rip the material out when expansion of the deck occurs. This product has better elastic properties than the previously used rubberised bitumen. Reasonable performance has been found though it tends to expand greatly when heated, and a slightly stiffer and less elastic product would be a better option.

As spans increased, so did the width of expansion joint, and compression seals were introduced to cater for the increased range of movements expected. The earliest seal used was the neoprene hose but this product proved to be inelastic and often fell through the joint leaving it completely open. 'Wabo' compression seals were then used firstly between steel angle nosings, and currently between fibre reinforced concrete nosings. One problem with this seal is that it can tend to debond from the concrete deck or steel and gradually work its way to the top of the joint where traffic damages the seal or, in some cases, rips the seal completely out. Steel angle nosings are also susceptible to damage through high impact loads imposed by vehicle tyres, especially where dry packed mortar has been rammed beneath the angle. This mortar breaks up under impact and the resulting loss of support leads to failure of the bars anchoring the plates into the concrete deck. The angles then start rattling and moving under load which cracks the bitumen at the edge of the angle.

'Alustrip' expansion joints are now commonly used and these consist of a thin neoprene sheet anchored into aluminium blocks which in turn are bolted down to the deck. These blocks can break loose if bolting was provided via cored holes rather than bolts cast into the deck. The seals also can become damaged and require resealing.

On large span bridges steel finger plates and steel sliding plate joints have been used. These joints have never offered a seal to moisture penetration and the sliding plates continually vibrate loose causing a danger to traffic. They have been superseded by heavy duty rubber joints such as 'Transflex', 'Waboflex' and 'Felspan'. A problem with these joints is possible debonding of the metal and rubber sections.

Epoxy mortar nosings were used in the past to support the joints but these thin sections, cast after the deck, only broke up under repeated impact loads.

On bridge decks with small movements and a large asphalt cover a product called 'ThormaJoint' has been used. This joint consists of a hot mix of selected stone and an elastomer modified bitumen binder, and looks like a strip (approximately 500 mm wide) of very dark asphalt. Performance is generally very good where it has been used, and provides improved ride over the joint.

3.5.2 Bearings

A large number of different bearing systems have been used in the past and only the more common types will be discussed here. The first precast and cast insitu beams usually sat on the headstocks with the only form of bond breakers between the two being a layer of clear grease, a sheet of malthoid or in some cases a sheet of lead. Locating dowels from the headstock were used but these have simply tended to break out the ends of the concrete beams, or in some instances, break out the top of the crosshead beneath the beam due to movement and edge loadings.

Mortar pads have had considerable use in the past and are generally found in good condition, though some rammed mortar pads beneath the beams tend to crack badly and spall the mortar.

Also to be found on many bridges are steel base plates on which rest the smaller steel bearing plates of the beams. Sometimes a phosphor bronze sliding plate may be inserted between the steel plates to aid in longitudinal movement between them.

Cast iron bearing blocks with sliding plates or pins, mild steel rollers and rocker bearings have also been used where large steel beam spans are present. Dirt, grit and corrosion due to moisture are a continual problem with these bearing types, with many rollers and rockers being completely seized up by corrosion.

Large span, heavy concrete bridges such as box girders can be supported on pot bearings or bearings with a P.T.F.E. (teflon) sliding disc. These are specialised high load bearings but the position of the P.T.F.E. strip should be noted, especially as it can tend to be squeezed out by vibration. Excessive rotations of the bearings should also be noted.

The most common bearing in use today is the elastomeric bearing in two different forms; as a 25 mm thick neoprene pad, or as a larger depth bearing with metal shim plates between elastomeric layers. The thinner bearing strips usually support small span beams and have few problems although if the bearing pedestals are poorly constructed then some areas of the pads may not carry load. The larger bearings can suffer from irregular bulging and shearing of the elastomeric/metal shim surface if poorly designed or manufactured. Rotation and shear of the bearings can occur with bridge movement, and this can cause lift-off of the bearings at the edge, and hence over-stressing at the opposite edge.

A common problem associated with large bearings is poor uneven pedestal construction resulting in significant areas of the bearing pads being unstressed.

Creep, shrinkage and elastic shortening due to post-tensioning in some structures cause shear stresses on the bearings. These bearings should be reset by jacking the structure, but this is rarely done unless shear is excessive. Slippage of the bearings can also occur in girder structures where retainers on the sole plates were not provided, with bearings working their way forward from the support area.

3.5.3 Damage due to accidents

The most common components affected by vehicular impact are barriers, kerbs, footpath slabs and end posts which can be severely abraded, spalled or damaged. Damage is usually self-evident.

Other areas that can be affected are columns, outer beams or soffits of overpass structures. Steel beams are particularly susceptible to damage from over-height vehicles which can cause severe deformations to the bottom flange or web of the member.

Bridges over navigable waterways may also have damage to pier columns and pile caps due to impact of vessels. The damage may be sufficient to cause major structural damage or movement of the column requiring an assessment of the structural adequacy of the bridge, or cause abrasion and spalling of concrete which can result in eventual corrosion of reinforcement.

3.5.4 Drainage

Ineffective drainage may affect a structure in several ways:

- Standing water in the carriageway which may create a serious traffic hazard.
- Debris carried by drainage flows will build up in areas, retain moisture, and promote corrosion.
- Leakage through deck joints and cracks will cause unsightly staining of beams, piers and abutments.
- Water flowing uncontrolled over concrete or steel surfaces or bearings below deck level may result in corrosion or unsatisfactory performance of bearings.
- Inadequate collection of drainage from decks and approaches can cause erosion, piping and washout or scour of the approach embankment and batter slopes along with undermining of foundations, particularly in areas where flows are concentrated at the end of the bridge around the end post and at ends of kerbs or service ducts. These areas should be inspected particularly after heavy rain or flooding.

- Inadequate or blocked weepholes in retaining structures can result in a build-up of pore water pressure, increasing lateral pressures on the wall.

3.5.5 Debris

The build-up of debris on the upstream side of structures can cause the following adverse effects:

- Impose lateral loads on a bridge superstructure during flooding for which it was not designed.
- Cause blockage of the waterway during flooding which can exacerbate problems of scour, undermining of foundations, flooding and in extreme cases total blockage and diversions of the watercourse.
- Development of permanent ‘obstructions developing within the waterway.

Build-up of debris is dependent on upstream catchment conditions/use and its impact is usually most severe in structures with small openings or low freeboard.

Additionally, the build-up of debris below a structure may become a fire hazard, increasing the risk of fire damage to piles and headstocks.

3.5.6 Vegetation

Uncontrolled and excessive growth of vegetation beneath, adjacent to or on a structure can have both positive and negative consequences. In the case of bridges and culverts, the presence of vegetation is typically negative as it can result in:

- fire hazard
- blockage of waterway
- retention of moisture around key components (bearings, girders, headstocks, etc.)
- build-up of leaf litter across decks will block scuppers and promote decay
- root ingress and growth can displace and damage components
- limited access to and poor visibility of structure components
- limited sight distances across structures
- overhanging limbs pose a hazard to road users.

For retaining structures, the presence of vegetation on the up and down slope will typically contribute to slope stability as well as improving resistance to erosion however large trees can become unstable and cause substantial damage if they fail. Vegetation growth on retaining structures limits access and visibility of components and, as for bridges and culverts, root ingress and growth can displace/damage components.

3.5.7 Waterway scour

Scour in watercourses and drainage paths has the potential to cause significant damage to the environment and engineering infrastructure.

Scour is the erosive action of flowing water, resulting in the removal, and subsequent deposition, of material from the bed and banks of streams and from around the piers and abutments of bridges.

Scour can result in a general lowering (degradation) or raising (aggradation) of the river bed, lateral erosion of river banks or the development of localised scour holes around piers/piles and abutments.

It is a requirement of Level two inspections to capture the waterway profile for bridges and culverts crossing waterways. This allows for the monitoring of general waterway behaviour over time.

Indications of recent 'fresh' scour activity include:

- loss of vegetation/exposed roots on waterway bank (refer Figure 3.5.7a)
- exposed piles (refer Figure 3.5.7b and Figure 3.5.7c).
- undermining/voiding to toe of bank or provided batter protection
- evidence of batter protection settlement such as cracking in rigid protection or slumping in flexible/semi-rigid protection (refer Figure 3.5.7d)
- loss of abutment batter protection
- undermining/voiding beneath abutment/piers
- accumulation of loose sediment adjacent river banks around piers (the latter may be indicative of localised scour holes being backfilled with loose material as water levels recede).

Degradation and localised scour can result in progressive settlement or movement of abutments, piers, culverts and any other structure in or adjacent to the waterway which, if not rectified can lead to total failure of the structure.

Figure 3.5.7a – Example of recent bank erosion

Figure 3.5.7b – Localised scour around pier pile cap and piles

Figure 3.5.7c – Localised scour around pier piles and debris build-up

Figure 3.5.7d – Settlement of rigid batter protection

3.5.8 Movement of the structure

Unanticipated movement of structures may result from:

- general scour of the stream bed in the vicinity of the structure
- local scour of the stream bed at piers or abutments
- movement of the ground due to land slips at or around the structure
- excessive earth pressure caused by movements or settlements of retained fill
- founding of structures on expansive clays
- collisions, in the case of bridges over navigable waterways, roads, or railways
- ‘freezing’ up of bearings or expansion joints.

Movements can usually be detected by observing:

- total closure or excessive opening of deck expansion joints
- bearing or jamming up between the end of the superstructure and abutment ballast wall with associated cracking and spalling
- cracking or excessive settlement of the approach embankments or heaving at toe
- undermining of foundations
- rotation of columns, piles, walls or adjacent poles, fences, etc.

It is important to report any of the above defects if they are observed as any movement could continue over a period of time and comparisons with past and future inspections is important to assess whether it is continuing, seasonal or has ceased.

3.5.9 Condition of approaches

The purpose of the approach embankment is to provide a stable transition between the bridge and adjacent pavement. Often it is also required to provide horizontal, and sometimes vertical support for the abutment foundation.

The most common defect of approach embankments is usually excessive settlement adjacent to the bridge abutment which causes unsatisfactory riding quality and possible damage to deck and expansion joints.

This can be caused by poorly compacted embankment, and or continuing settlement of the underlying ground. Instability of ground and embankment can also be observed in its early stages by excessive settlement or movement of the embankment.

It should be noted that while the subsidence behind bridge abutments is often attributed to settlement of embankment fill the defect may often be caused by other factors including:

- settlement or rotation of walls which allows loss of embankment material generally as a result of leaching of fines
- settlement of infill panels or backing slabs, which generally occurs as a result of softening of moisture susceptible founding material or following scouring of the footing
- erosion, piping, washout and scour of the embankment, particularly after heavy rain or flooding, or due to inadequate or blocked drainage.

4 References

1. Ministry of Transportation, Ontario (1989). *Ontario Structure Inspection Manual*. Highway Engineering Division
2. Austroads (1991). *Bridge Management Practice*
3. Transit New Zealand (1991). *Bridge Inspection and Maintenance Manual*
4. Austroads (1991). *Guide to Bridge Construction Practice*
5. Austroads (2009). *Guide to Bridge Technology Part 2: Materials*
6. Austroads (2009). *Guide to Bridge Technology Part 4: Design Procurement and Concept Design*
7. Austroads (2009). *Guide to Bridge Technology Part 6: Bridge Construction*
8. Austroads (2009). *Guide to Bridge Technology Part 7: Maintenance and Management of Existing Bridges*
9. Bootle, K. R. (1983). *Wood in Australia, Types, Properties and Uses.* McGraw Hill: Sydney
10. CSIRO, Division of Building (1975). *Effect of fire on Timber Engineering.* Melbourne, Lectures given by Officers of CSIRO Div. of Building Research Highett
11. McGregor, K. (1991). *Guidelines for Timber Bridge Inspection, Maintenance & Repair.* VicRoads
12. VicRoads (2014). *Road Structures Inspection Manual*, Principal Bridge Engineer’s section, VicRoads Technical Services, VicRoads
13. VicRoads (2011). *Technical Note 102 Fire Damaged Reinforced Concrete – Investigation, Assessment and Repair*, VicRoads Technical Services, VicRoads