Manual

Structures Inspection Manual
Part 1: Structures Inspection Policy

September 2016

Queensland Government

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TMRStructuresInspectionManual@tmr.qld.gov.au

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1 Objectives

The purpose of this policy is to ensure that the condition of all qualifying highway structures is systematically monitored to ensure that conditions which may lead to severe structural damage or collapse are identified as soon as possible in order that the appropriate intervention or remedial action may be undertaken.

In addition, the data collected from the inspections may be used to:

- develop inspection and maintenance programs
- carry out load capacity assessments
- provide feedback to the design process
- monitor the effectiveness of maintenance treatments on a local or state-wide basis
- maintain hazard registers including the department's Asbestos Register.

The policy also identifies accountabilities for structures asset management and establishes the requirements for data management and a systematic inspection and condition rating programme.

2 Structures Asset Management System (SAMS)

The SAMS has been developed to ensure that the department's structures are managed effectively and efficiently. Inspection and condition rating is an integral component of the SAMS and its relationship with other principal components of the system is shown in the system framework diagram (refer Figure 1) and in the mechanisms used to deliver desired outcomes (Figure 2).

The primary objective of the SAMS is to establish an integrated and accessible information system for structure inventory, condition, load capacity and inspection and works history.

The Bridge Information System (BIS) has been developed for this purpose.

3 Scope

3.1 Asset types

This policy applies to the following structures:

- All bridges
- All 'major' culverts. I.e. meeting the following criteria:
  - metal culverts (steel and aluminium):
    - at least one barrel (cell) with span, height or diameter ≥ 1.2 m, or
  - all other culverts:
    - pipes with at least one barrel (cell) with diameter ≥ 1.8 m, or
    - rectangular/oval/arch culverts at least one barrel (cell) with span > 1.8 and height > 1.5 m.
    - stock and pedestrian underpasses

These culverts typically have an opening large enough to:

- access without specialist equipment and are therefore capable of being inspected relatively easily.
- close the road and create a significant safety hazard in the event of structural failure.

Further to the above, the following asset types are also included, irrespective of ownership, where there will be significant impact to public safety or critical network function in the event of failure:

- tunnels
- busway bridges, including elevated and underground stations
- pedestrian overbridges at busway stations
- retaining structures[1] below the highway, including gravity, cantilever and mechanically stabilised earth structures > 1.5 m high within 1.5 x retained height of the carriageway shoulder
- retaining structures[1] above the highway, including gravity, cantilever and mechanically stabilised earth structures > 2.0 m high within 1.0 x retained height of the carriageway shoulder
- gantries over road network and all structures supporting signs over traffic lanes. This includes large signs with truss supports in close proximity to traffic lanes (i.e where failure of the supporting structure could impact on road users/network function)
- pedestrian/cycle bridges over the road network
- rail/light rail bridges over the road network
- high risk ‘minor culverts’ e.g. culverts falling just outside of criteria listed above may be referred to BCMAM by the District for consideration for reclassification as a major culvert if failure were to result in a significant impact on public safety or critical network function.

[1] Retaining structures at bridge/culvert sites should be recorded as a component of that structure.

Where the department do not own the structure, or are not responsible for its inspection/maintenance under a formal agreement, any decision to include the structure on a formal inspection program is to be considered on a case-by-case basis and shall take into account the department’s confidence in how the structure is being managed. The asset owner shall be consulted.

Notwithstanding the above, all structures meeting the above criteria shall be captured in the BIS.

4 Structure identification

All structures complying with these criteria shall be allocated a unique number in the BIS and, in addition, shall be physically numbered to permit ready identification in the field.

There is capacity within the BIS which permits Districts to record data on "other" structures if desired, (e.g. minor culverts). In this event the Districts can adopt a local numbering system to locally manage these assets. It is anticipated that these smaller structures shall be managed through the RMPC system.

Figure 1 – Structures Asset Management System Framework

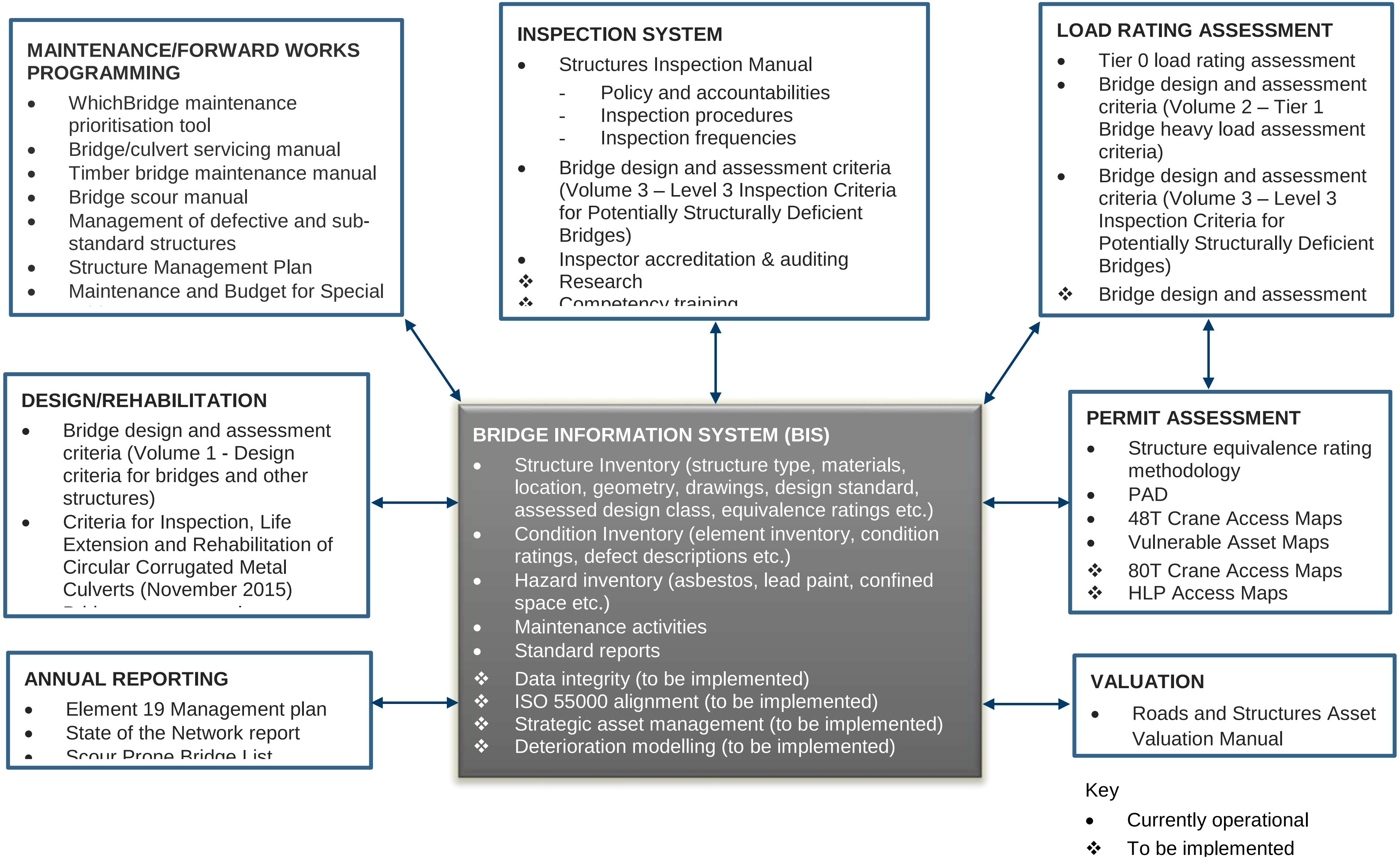
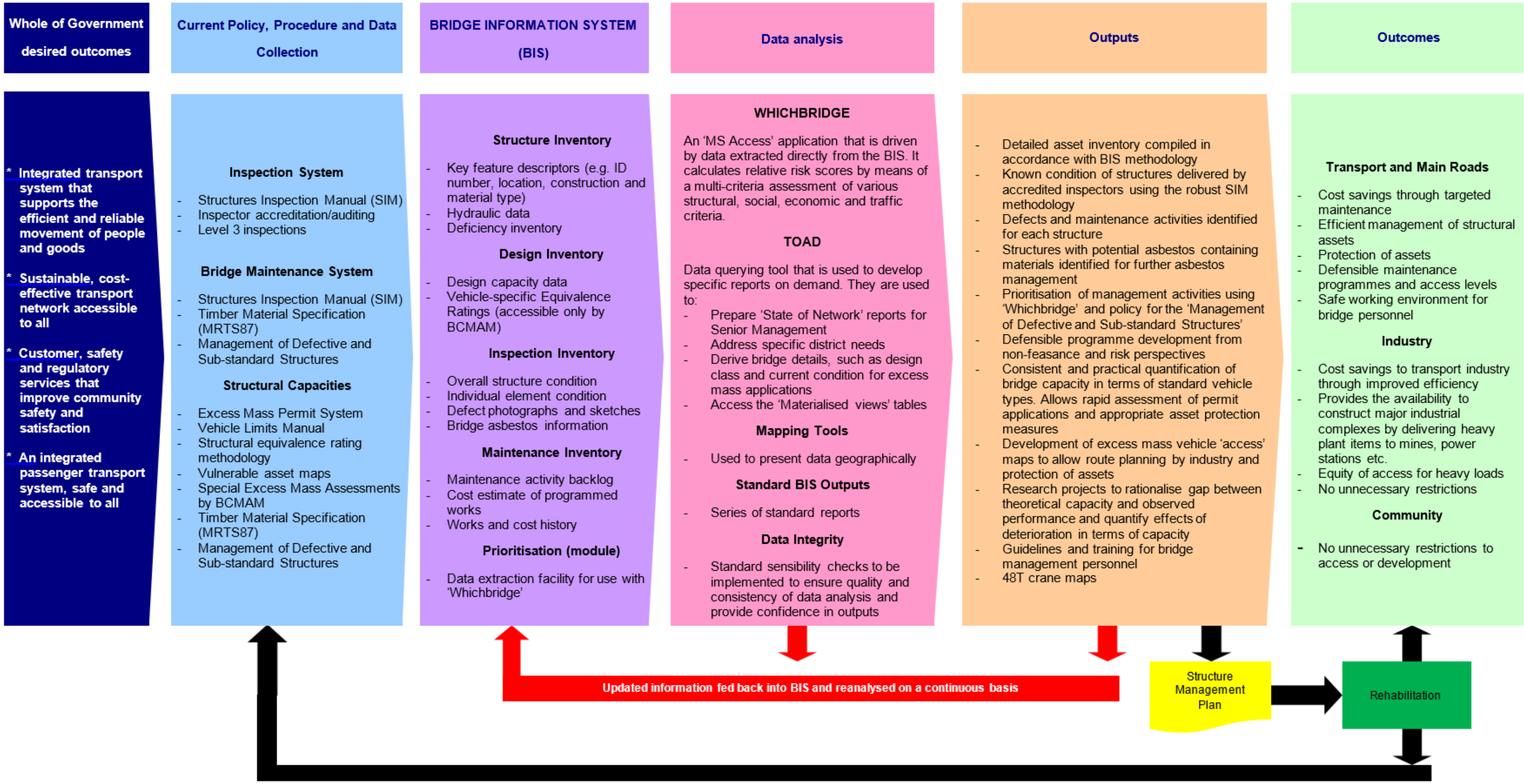


Figure 2 – Structures Asset Management Mechanisms



5 Accountability

5.1 General Accountabilities

District Directors are accountable for the management of all highway structures on the State controlled road network. These management responsibilities include:

- Monitoring the delivery of the structure inspection programme.
- Ensuring that Routine Maintenance Inspections are carried out at least once every 12 months, inspection data is monitored and recorded and recommendations are actioned.
- Ensuring that Condition Inspections are carried out at the required frequencies, inspection data is monitored and recorded and recommendations are actioned.
- Ensuring that the required "Maintenance Activities" are recorded, costed and entered in the BIS and managed effectively.
- Commissioning Detailed Engineering Inspections, investigations and analysis when required, and ensuring that recommendations are actioned.
- Ensuring that all inspection data is transferred to the Bridge Information System within 30 days of its collection. However, in the event that a defective structure is detected, all inspection data shall be entered into the BIS within seven days of its collection.
- Development of "Structure Management Plans" in accordance with the guidelines in Appendix F. Plans are to be developed in conjunction with Structures Directorate, for all defective or substandard structures.

Deputy Chief Engineer (Structures) through **Director (Bridge Construction Maintenance & Asset Management (BCMAM))** is accountable for:

- Promulgating and monitoring the implementation of this policy.
- Developing, implementing and maintaining the Bridge Information System (including updating descriptive and design data) and providing the necessary access and reporting mechanisms for all TMR personnel involved in structures management.
- Ensuring the technical adequacy of the specified inspection processes.
- Developing and supporting the technical procedures; including the preparation of the supporting manuals and the training and accreditation programmes necessary to implement this policy.
- Monitoring the delivery of the inspection programme through a data and physical auditing programme.
- Accreditation of inspectors and maintain an accredited inspector register.
- Assist Districts in the development and certification of Management Plans for substandard and defective structures.
- Supporting the BIS Functional Manager – Director (BCMAM).
- Providing resources to maintain and audit data that is held in the BIS.
- Supplying the specialist resources to enable Director (BCMAM) to develop, implement and support the Structures Inspection Manual and attendant procedures and processes. This

includes BCMAM arranging or carrying out detailed structural engineering inspections for the Regions and Districts.

Executive Director (Program Management Delivery) is accountable for:

- Development of uniform, consistent and cost effective inspection programmes; including quality assurance systems and the co-ordination of joint services among regions and districts.

Executive Director (Strategic Investment and Asset Management) is accountable for:

- Providing resources to develop and maintain the BIS IT system through the ARMIS service request (ASR) system.
- Providing resources to train and support BIS users.

5.2 Overview of Responsibility for WHS in Inspections

All inspections must comply with the requirements of:

- this manual
- any applicable legislation, codes of practice, standards and Transport and Main Roads policy/manuals including but not limited to:
  - *Work Health and Safety Act 2011*
  - *Work Health and Safety Regulations 2011*
  - *Bridge Asbestos Management Plan*

All inspectors are responsible for their own personal safety and that of others impacted by inspections at all times.

The department demonstrates part of its duty of care for those undertaking inspections and the general public by:

- Providing a generic list of hazards typical for many inspections.
- Providing known specific hazards for each structure in the structure information passed onto inspectors (e.g. presence of possible/confirmed asbestos containing materials).
- Requiring a minimum of two people on site at any given time when inspecting.
- Requiring inspectors to provide and submit a Safe Work Method Statements (SWMS) for each inspection (or set of inspections) undertaken for review and comment before commencing site works.
- Conducting audit and surveillance to ensure inspections are carried out as per the SWMS.
- Requiring inspectors to submit an update of the specific road structures inspection hazards to TMR following site inspections.

If an inspection is required from water, any vessel used for the purpose and its operation will be required to satisfy the legal obligations of the Marine Act, other relevant Acts, and associated regulations.

Where inspections are carried out on structures located over or under the assets of other Authorities, the relevant regulations and Codes of Practice relating to work on or close to their assets must be adhered to.

6 Structure Information

Comprehensive inventory and condition data is captured in the BIS, which is maintained by the Deputy Chief Engineer (Structures). This system provides accessible and timely information to all TMR personnel involved in structures management and is integrated with ARMIS. This connects all related structure and road data through a common location reference system.

The District Director will act as an agent for the Deputy Chief Engineer (Structures) and is responsible for entering and managing the inventory, inspection, condition and maintenance data at the local level in accordance with the documented guidelines for the BIS and this manual.

Details of the data capture requirements for the various inspection levels are defined in the inspection requirements section.

7 Inspection Requirements

The safety and condition of structures on the state road network is monitored through a three level hierarchical inspection regime that was introduced in March 1998.

The overall requirements for each inspection level are covered in *Part 3 – Procedures*.

The three inspection levels are as follows:

- Level 1 – Routine Maintenance Inspection
  - A visual inspection to check the general serviceability of the structure, particularly for the safety of road users, and to identify any emerging problems.
- Level 2 – Condition Rating Inspection
  - An inspection to assess and rate the condition of a structure (as a basis for assessing the effectiveness of past maintenance treatments, identifying current maintenance needs, modelling and forecasting future changes in condition and estimating future budget requirements).
- Level 3 – Special Inspection
  - An inspection to provide improved knowledge of the condition, load capacity, in-service performance or any other characteristic beyond the scope of other types of inspection. Special inspections may be used to inform/develop the scope of other types of inspection. Level 3 inspection categories include:
    - structural engineering
    - ACM identification
    - ACM verification
    - underwater access
    - fracture critical/low redundancy
    - sub-standard load rating
    - complex/unique structures
    - known/suspected deficiencies
    - confined space inspection.

Manual

Structures Inspection Manual
Part 2: Deterioration Mechanisms

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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.

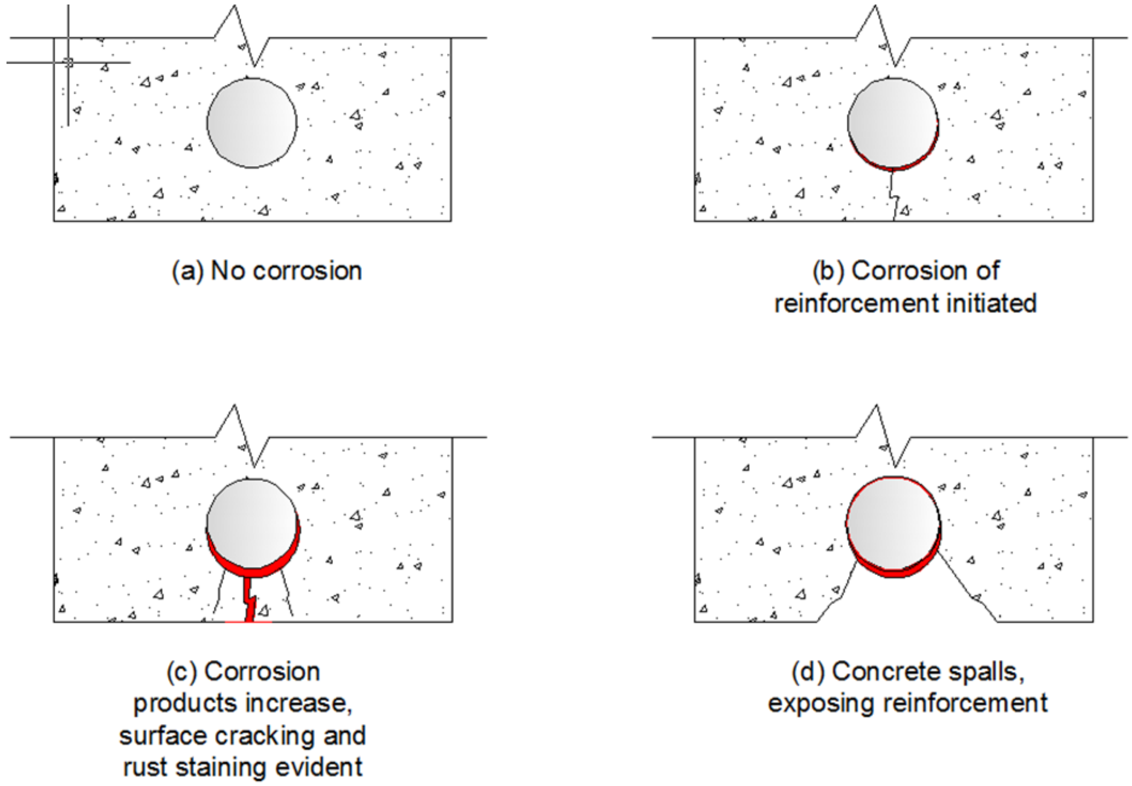
[2] This phenomena results in cracking of concrete and also promotes other deterioration mechanisms.

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.

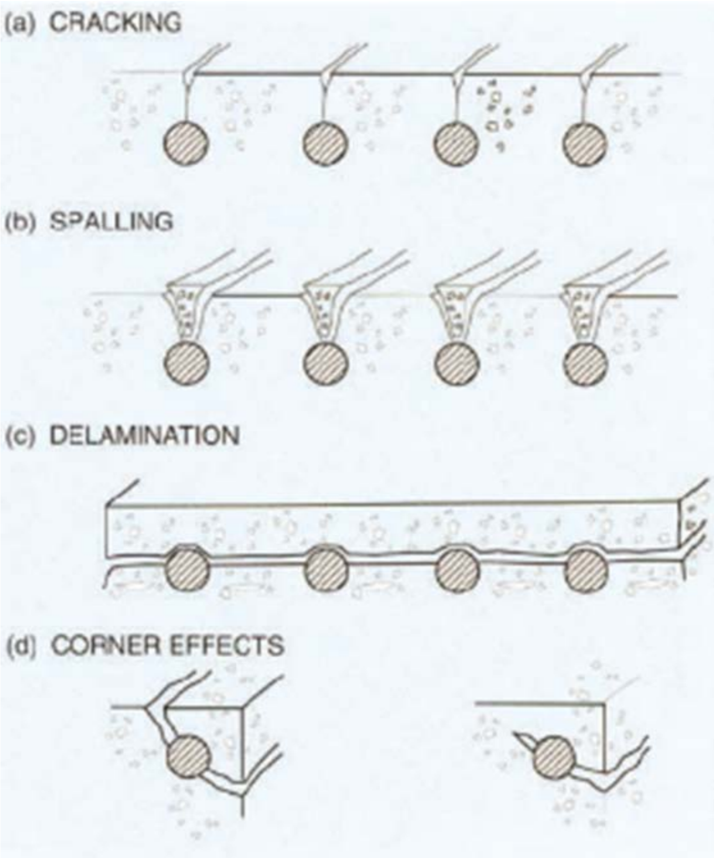
Figure 2.2.1a – Stages of reinforcement corrosion



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



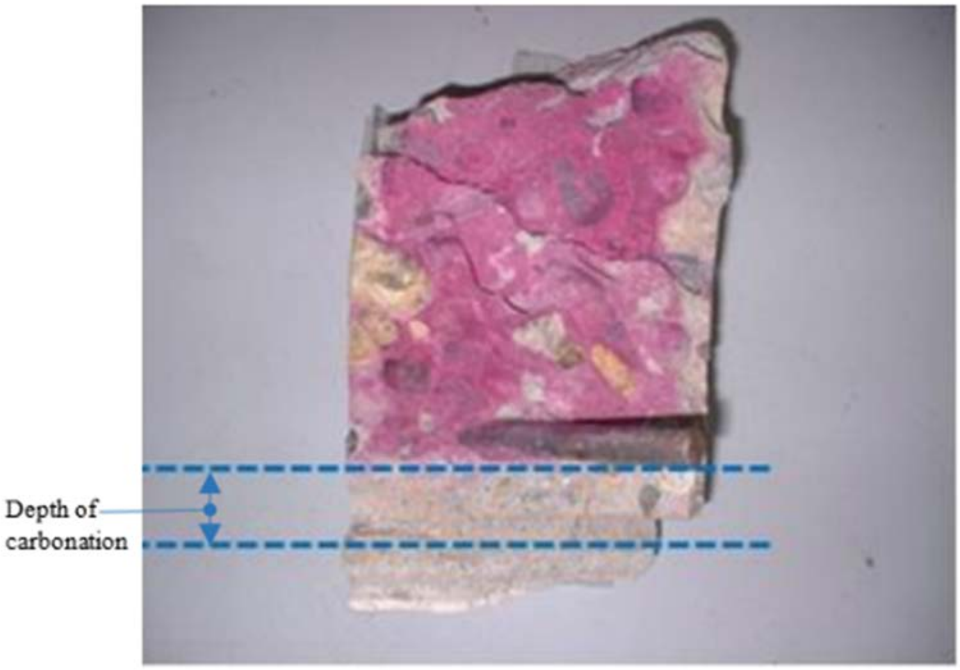
In Queensland, the most common example of damage resulting from reinforcement corrosion is found in the square section reinforced concrete piles which were used extensively until the introduction of prestressed octagonal piles. Cracking typically follows the line of the corner reinforcement where two 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

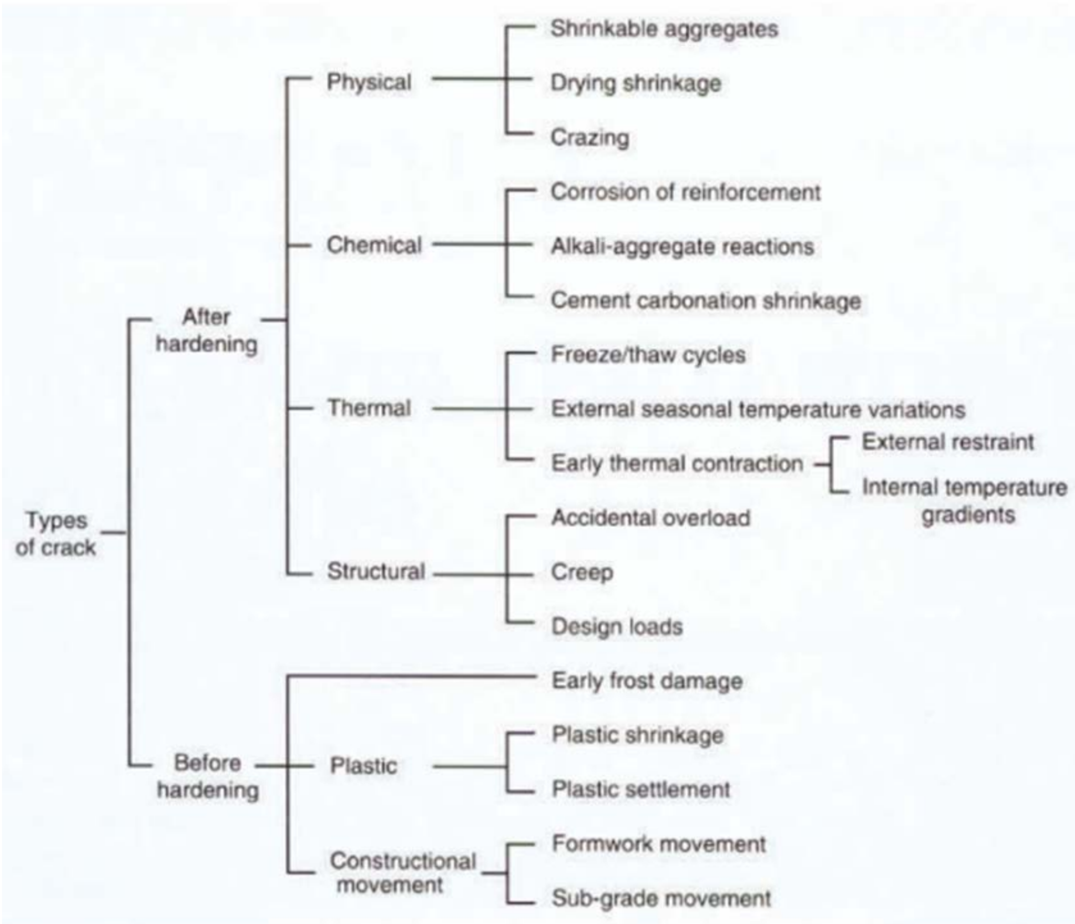
2.2.3 Cracking

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

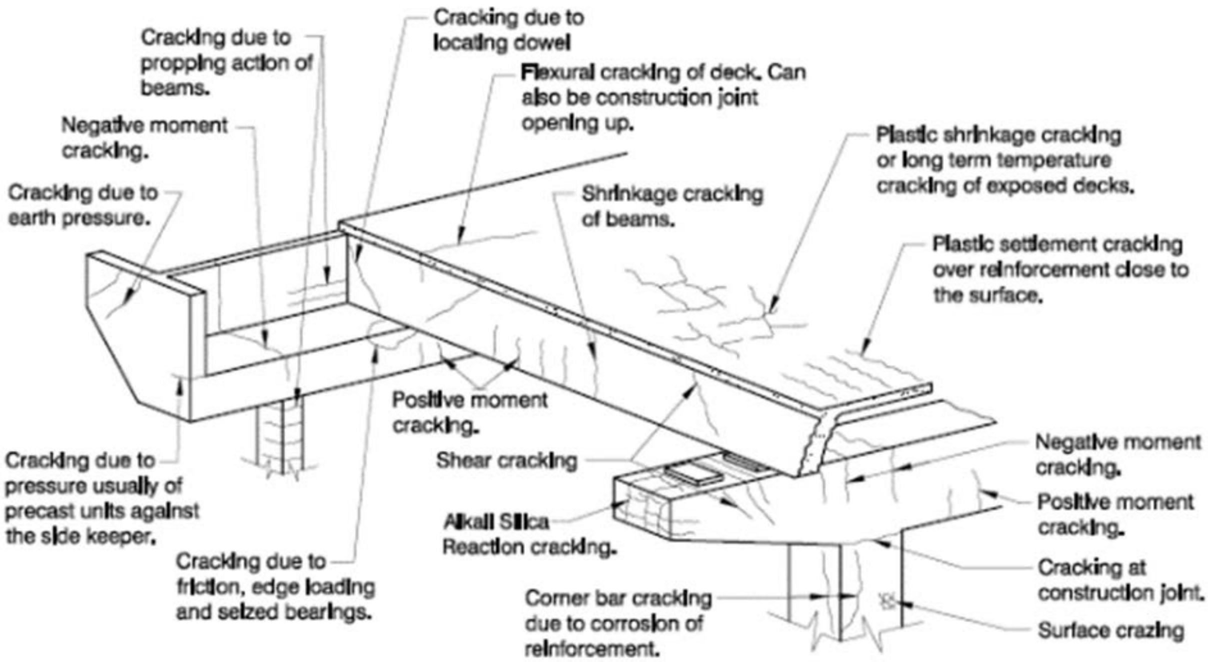


2.2.3.1 Plastic cracking

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.

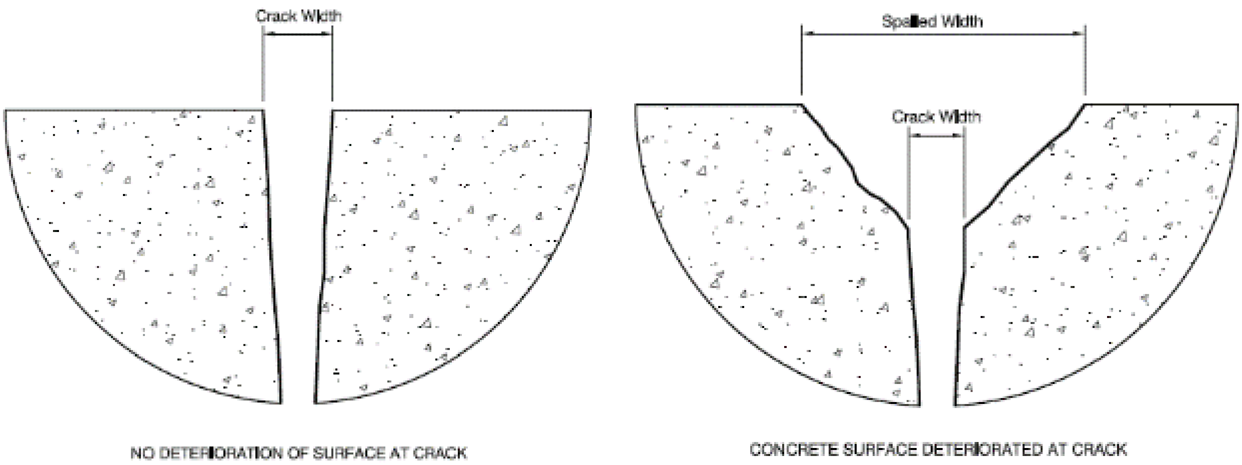
The location and orientation of cracks is usually a good indicator of the likely causes.

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

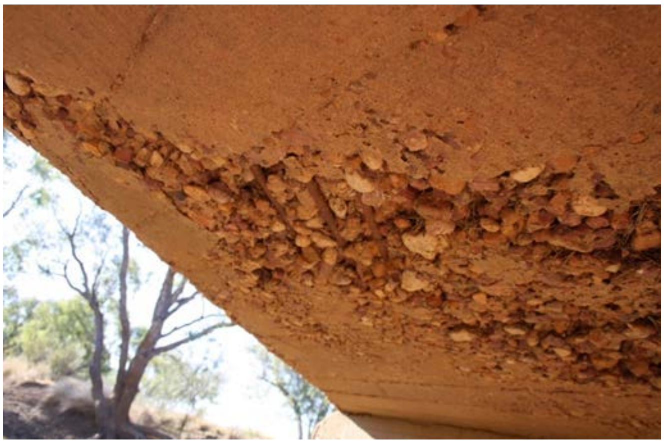


Figure 2.2.6f – Abrasion of concrete

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

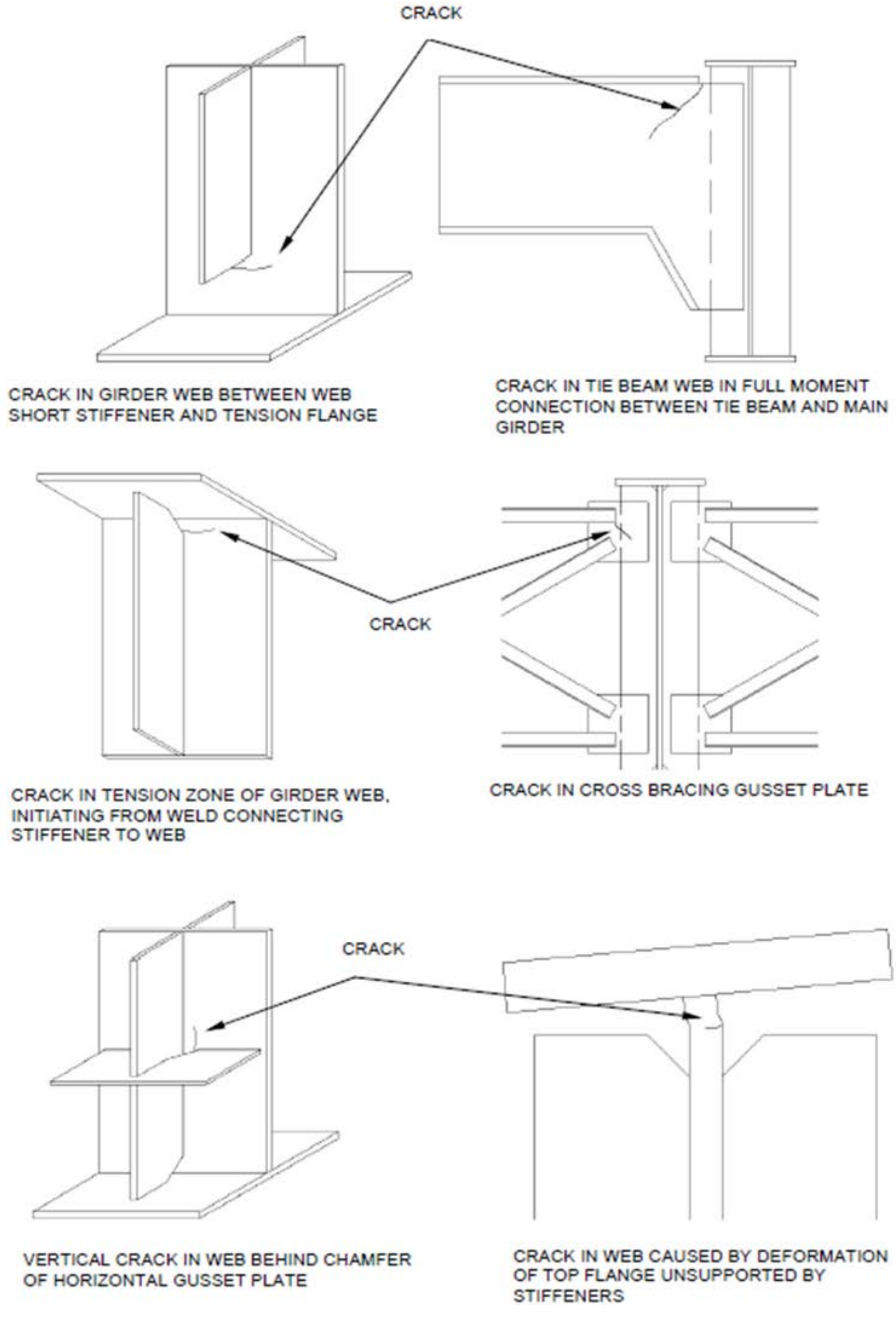
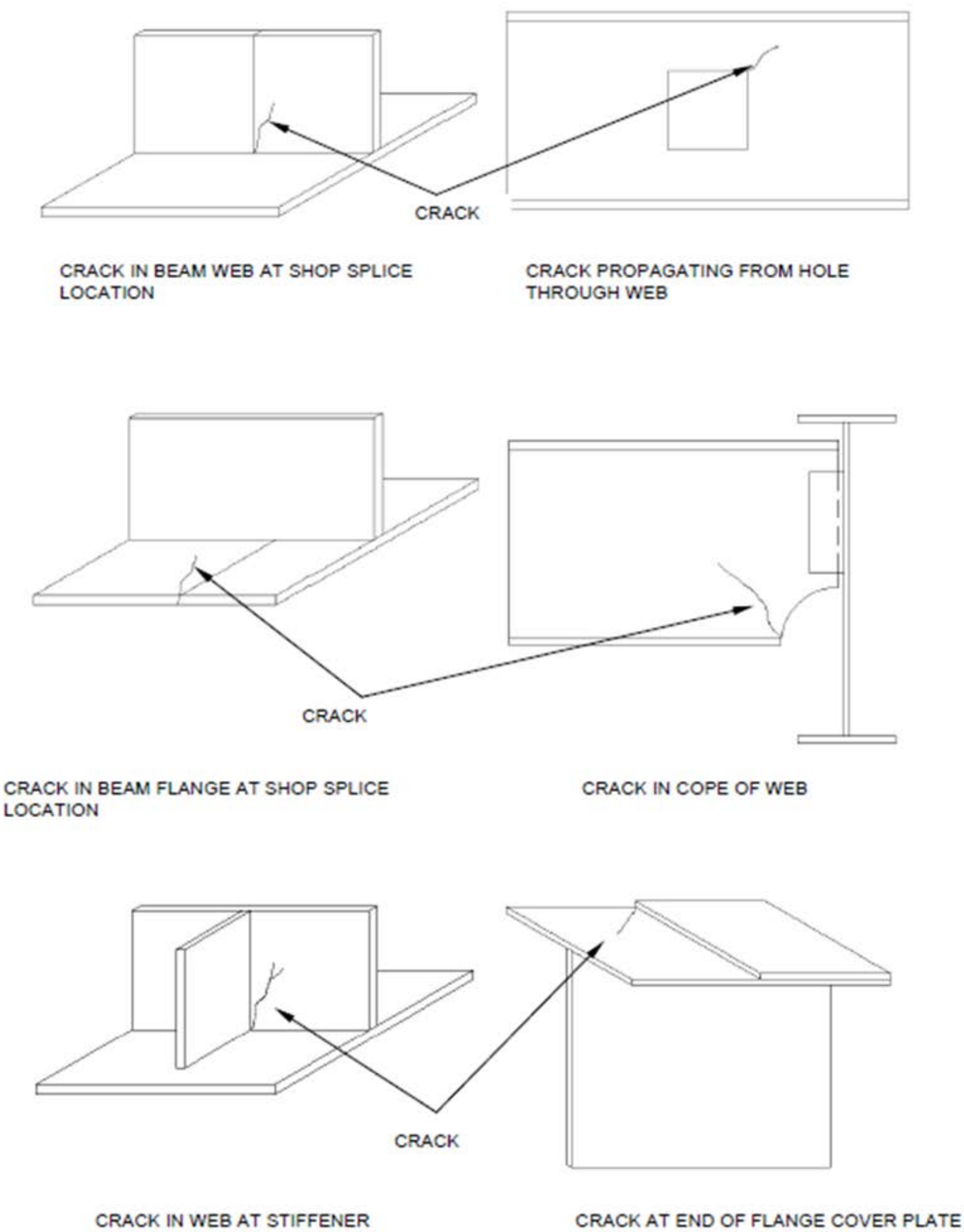


Figure 2.3.4d – Common crack locations in steel



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)

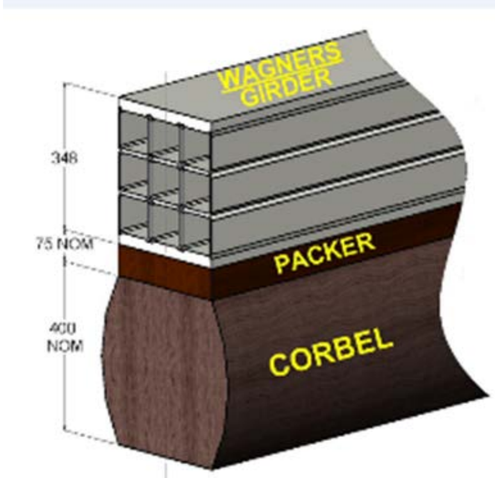


Figure 2.6.2b – Loc Composite Hybrid girder (1st generation)

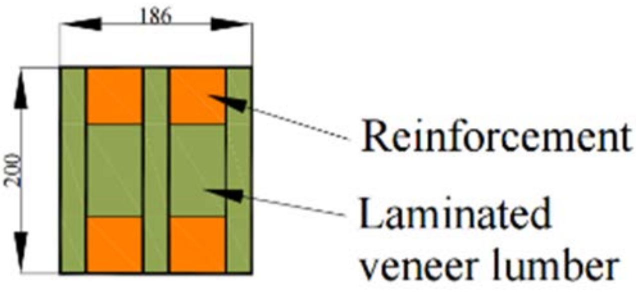


Figure 2.6.2c – Wagners FC girder (2nd generation – WCFT S1, S2 & S3)

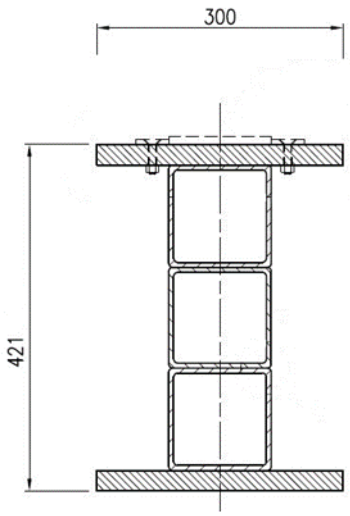
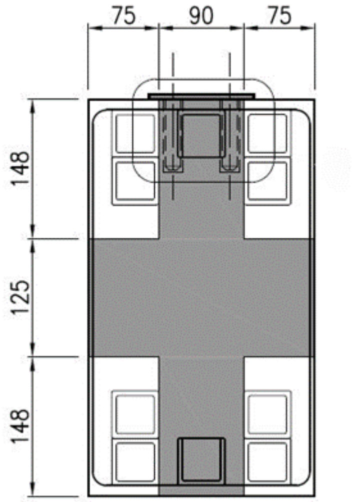


Figure 2.6.2d – Loc Composite Hybrid girder (2nd generation -LOC 400 & LOC 420))



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

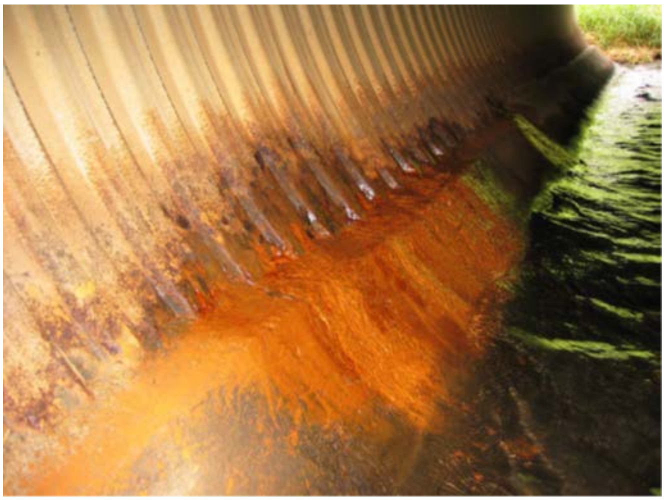
3.4.2.2 Buried corrugated metal culverts

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.

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Manual

Structures Inspection Manual
Part 3: Structures Inspection Procedures

September 2016

Queensland Government

Feedback: Please send your feedback regarding this document to:
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Contents

Tables

Figures

1 General

1.1 Levels of inspection

The Transport and Main Roads *Structures Inspection Manual Part One – Structures Inspection Policy* has identified the need for a systematic program of inspections based on three levels of inspection.

The three inspection levels are as follows:

- Level 1 – Routine Maintenance Inspection
  - A visual inspection to check the general serviceability of the structure, particularly for the safety of road users, and to identify any emerging problems.
- Level 2 – Condition Rating Inspection
  - An inspection to assess and rate the condition of a structure (as a basis for assessing the effectiveness of past maintenance treatments, identifying current maintenance needs, modelling and forecasting future changes in condition and estimating future budget requirements).
- Level 3 – Special Inspection
  - An inspection to provide improved knowledge of the condition, structural capacity, in-service performance or any other characteristic beyond the scope of other types of inspection. Special inspections may be used to inform/develop the scope of Level 1 and Level 2 inspections. Level 3 inspection categories include:
    - structural engineering
    - asbestos containing material (ACM) identification
    - ACM verification
    - underwater access
    - fracture critical/redundancy
    - sub-standard load rating
    - complex/unique bridges
    - known/suspected deficiencies
    - confined space inspection.

1.2 Safety

All inspections must comply with the requirements of:

- this manual
- any applicable legislation, codes of practice, standards and Transport and Main Roads policy/manuals including but not limited to:
  - *Work Health and Safety Act 2011*
  - Work Health and Safety Regulation 2011
  - MRTS96 *Management and Removal of Asbestos.*

While structure inspections, detailed investigations and maintenance activities may not be categorised as a 'specified work' category, documented safe work procedures (for example, Safe Work Method Statements) shall be prepared in order to:

- reduce the risk to staff undertaking field work
- provide documentary evidence that the department has fulfilled its obligation as an employer under the Act.

If inspection is required from water, any vessel used for this purpose and its operation will be required to satisfy the legal obligations of the *Transport Operations (Marine Safety) Act 1994*, other relevant Acts, and associated Regulations.

Where inspections are to be carried out on structures located over or under the assets of other Authorities, the relevant Regulations and Codes of Practice relating to work on or close to their assets must be adhered to and, where necessary, referred to in the safe work procedures developed for the inspection.

1.3 Component designation

General terminology used to label the components of various structure types, irrespective of the level of inspection being undertaken, is shown in Appendix B and Appendix C.

Component location and numbering shall be in accordance with the procedures outlined for Level 2 inspections in Section 3.

1.4 Manual updates

The department's Bridge Construction, Maintenance and Asset Management (BCMAM) section will update the manual at regular intervals (at least every 12 months) to reflect feedback received and any changes to inspection practice.

A feedback register advising of all current known issues is maintained on the *Structures Inspection Manual* web page. All those required to use the Manual, including inspectors, should check the register at regular intervals for issues/updates.

2 Level 1 – Routine Maintenance Inspection

2.1 Purpose

The purpose of a Level 1 inspection is to check the general serviceability of the structure, particularly for the safety of road users, and identify any emerging problems.

Level 1 inspections may be carried out in conjunction with routine maintenance of the structure and the adjacent pavement.

2.2 Scope

The scope of a Level 1 inspection includes:

- inspection of approaches, waterway, deck/footway, substructure, superstructure and attached services to assess and report any significant visible signs of distress or unusual behaviour, including active scours or deck joint movements
- recommendation of an exceptional Level 2 inspection or a Level 3 inspection if warranted by observed distress or unusual behaviour of the structure

- identification of maintenance requirements that fall outside the expertise and/or available material and equipment resources at hand
- verification of the 'Structural Inventory' data held in the Bridge Information System (BIS) as part of the initial inspection and where requested thereafter (**Structural Inventory Verification forms** are available from the BIS for this purpose).

2.3 Extent of inspections

The Level 1 inspection is a visual inspection which may be carried out in conjunction with routine maintenance of the structure and adjacent pavement and shall cover components above ground and water level listed in the procedure checklist.

Components that need not be inspected for Level 1 inspections are:

i. interiors of box girders or any other structure/component that constitutes a confined space

ii. areas behind abutments that are inaccessible

iii. piles and foundations below ground or water level

iv. piers and pier crossheads located in permanent water

v. components located in spans over permanent water

vi. items requiring special access equipment including boom lifts, underbridge access units, boats, ladders or scaffolding to perform the inspection (unless access is already being provided for routine maintenance requirements)

vii. all components above deck soffit level for bridges crossing over the road network.

These components will be inspected as part of a Level 2 or Level 3 inspection.

No access to the rail corridor is required for Level 1 inspection of road over rail bridges.

A visual assessment of items iv – vi should be made using binoculars where practical.

2.4 Inspection frequency

Level 1 inspections shall be carried out at the frequency specified in Table 2.4.

Table 2.4 – Level 1 inspection frequencies

Structure type	Overall condition state of structure	Inspection frequency (years)
Timber structures and steel culverts (in permanent standing water)	1 – 2	1[1]
	3 – 4	1[2]
Bridges and culverts[4]	1 – 2	1[1]
	3	1[1]
	4	1[2]
Tunnels	1 – 2	1[3]
	3	1[3]
	4	1[3]

Structure type	Overall condition state of structure	Inspection frequency (years)
Busway bridges, including elevated and underground stations and pedestrian overbridges at busway stations	1 – 2	1[3]
	3	1[3]
	4	1[3]
Other bridges over the road network	1 – 2	1
	3	1
	4	1
Retaining structures above/below the road network (excludes retaining structures inspected as part of any other structure)	1 – 2	1[3]
	3	1[3]
	4	1[3]
Large Traffic Management Signs (LTMS) and gantries	1 – 2	1[1]
	3	1[1]
	4	1[2]

[1] Generally not required in the same years as level 2 or level 3 inspections.

[2] Level 1 and level 2 inspections to be staggered by six months to ensure inspections occur every six months.

[3] Or at frequency specified by structure-specific maintenance manual (whichever is greater).

[4] Includes critical ‘minor’ culverts.

All annual inspections shall be completed no less than 10 months and no greater than 14 months after the previous inspection.

Further, a Level 1 inspection is required following major flooding events (bridges and culverts in affected waterways only), fire or accident damage or as recommended in a Structures Management Plan or by BCMAM.

2.5 Inspector accreditation

Inspections shall be conducted by personnel who have extensive practical experience in pavement and structures routine maintenance. They shall be competent to judge the visual condition of structures and the road approaches for visual defects.

Accreditation requirements for Level 1 inspectors are detailed in Appendix E.

2.6 Inspection procedure

2.6.1 Preparation for inspection

Prior to commencing inspections, the inspector shall ensure that all relevant documentation, inspection equipment and safety equipment is in place along with the appropriate arrangements with the relevant road, railway or other Authorities for temporary access as required to carry out the inspection. Safety plans must be prepared and approved.

2.6.2 Inspection

The Level 1 inspection may be carried out in conjunction with Routine Maintenance activities.

When the inspection is carried out as part of Routine Maintenance activities, the Maintenance Contractor shall attend and rectify items requiring attention within the scope and limitations specified in this procedure for plant, equipment and expertise.

Any major defects identified in the course of this inspection must be photographed and/or sketched and recorded on the **Photographs and Sketches Record (A1/2)**.

At the site, the inspector shall proceed in a systematic manner to check the applicable inspection items listed in Table 2.6.2.

Table 2.6.2 – Level 1 inspection items

Approaches	
Signs/delineation	completeness
	loose/missing fixings
	damage
	cleanliness
	orientation
Guardrail	correct height/alignment
	loose or missing fixings
	impact damage
	material deterioration (steel corrosion, timber decay and so on)
	damaged/missing spacer blocks
	connection to bridge barrier (the approach barrier and bridge barrier should preferably be interconnected; if not, note in ‘Comments’ section of report form)
Road drainage	debris/vegetation growth inside drains, channels, inlet/outlet pits and sumps which may obstruct free drainage
	leaking drainage pits/structures
	scour/erosion at drainage outlets, particularly adjacent to abutments/foundations, culvert outlets and deck run-off
Wearing surface	settlement
	depressions
	pot holes
	cracking
Surface over structure	
Wearing surface	settlement
	depressions
	pot holes
	cracking
Footways (if any)	unevenness/trip hazards

Approaches	
Drainage	accumulations of debris on the deck, in gutters, scuppers and drains which may obstruct free drainage
Bridge barrier	correct height/alignment
	loose or missing fixings
	impact damage
	material deterioration (steel corrosion, timber decay and so on)
	damaged/missing spacer blocks
	delineators for completeness, damage, cleanliness, orientation and visibility to road users
Deck joints	loose/missing fixings
	damaged/missing components
	dirt/detritus build-up in joints which may impede free movement
	leakage
	damage/deterioration of nosings
Embankments and waterways	
Embankments	erosion
	scour
	slope stability
Slope/batter protection	undermining
	settlement
	loss of material
Vegetation	bushes, trees within 2.0 m of abutments and wingwalls
	bushes/trees within waterway channel
	vegetation affecting sight distance onto/across structure
Waterway	accumulation of debris against or adjacent to structure
	localised scour adjacent to/beneath the structure
	lateral bank erosion adjacent to/beneath the structure
	channel degrading
	channel aggrading
	structure in permanent standing water

Approaches	
Substructure	
Abutments, piers, wingwalls, retaining structures and foundations	cracking
	splitting
	distortion
	movement
	steel corrosion
	vegetation growth in joints of coursed masonry/fascia panels
	weepholes for blockages affecting free drainage
	timber members for decay, termite activity, marine borer and other insect attack
Headstocks, bearing pedestals and substructure drains	accumulations of dirt and debris which may obstruct free drainage and cause ponding or restrict bearing movement
Bearings	corrosion
	excessive deflection/bulging
	delamination of elastomeric bearings
	damage to pedestals/plinths
	noticeable build-up of deposits of aggressive salts, silt, debris and bird or bat droppings
Superstructure	
Deck/girders	obvious evidence of spalling, cracking, staining, dampness or corrosion
	excessive movement/vibration under load
	for noticeable build-up of deposits of aggressive salts, silt, debris and bird or bat droppings
	blocked vent holes
	timber members for termite activity, rotting, marine borer and other insect attack
	timber members for excessive member deflections
	timber girders and corbels for excessive sniping
	loose joints and fasteners
	propping for tightness of wedges in deck cambering or temporary works
Culvert barrel	distortion/deflection of barrel
	invert corrosion/abrasion
	obvious evidence of spalling, cracking, staining, dampness or corrosion

Approaches	
Large Traffic Management Signs (LTMS) and gantries	
Footings	obvious evidence of spalling, cracking or reinforcement corrosion
	rotation/settlement
	erosion/undermining
Base plates, fittings and hold-down bolts	cracking, spalling or voids in mortar pad
	debris/fill over base plate
	corrosion of fixings/base plate
	loose/missing fixings and thread engagement
Columns	corrosion, buckling, bending, rupture, rotation or misalignment of sections
	impact damage
	verticality of members
	protective coating loss
	loose/missing fixings and thread engagement
Cantilever arms/gantry beams	corrosion, buckling, bending, rupture, rotation or misalignment of sections
	separation/distortion at joints/splices
	sagging of members
	protective coating loss
	loose/missing fixings and thread engagement
Signage and ancillaries	completeness, damage, cleanliness, orientation and visibility to road users
Miscellaneous	
Roadway beneath structure	delineation
	barriers
	road drainage
Services	location and condition of any services attached to or in close proximity to the structure
Appearance	graffiti

2.7 Structural Inventory data

In almost all cases, the Structural Inventory data held in the BIS will have been populated from the drawings when the structure details were first entered into the BIS and, as such, most or all of the fields will have values.

Prior to the inspection, the inspector shall confirm whether a Structural Inventory Verification form has been completed for the structure.

If not, the inspector shall print out a copy of the **Structural Inventory Verification** form and, while on site:

- where the data fields have been populated in the BIS:
  - compare details on the form with those present on site
  - where the populated data is correct the inspector shall initial the data (in the corresponding 'initial' box) to confirm it has been verified
  - where the populated data is incorrect, the inspector shall cross it out and enter the correct information (obtained from the appropriate reference table) in the corresponding box on the form and initial the data (in the corresponding 'initial' box) to confirm it has been verified.
- where the data fields have not been populated in the BIS:
  - fill in empty data fields with the correct value from the appropriate reference table and initial the data (in the corresponding 'initial' box) to confirm it has been verified. If it is not possible to determine or verify the details, then the box should be left blank. The reason for not being able to determine or verify the details should be recorded.

A copy of the form is included in Appendix A3.

2.8 Data recording

All data obtained from the inspection shall be recorded on the relevant **Level 1 Inspection Report (A1/1)**. These forms have been designed to meet the following objectives.

- assist the inspectors carry out and record an inspection within the scope of and to the extent required for this level of inspection
- record the defects and:
  - where the inspection is being conducted in conjunction with routine maintenance and rectification of the defect is covered by the Road Maintenance Performance Contract (RMPC) and is within the capabilities and resources available on the patrol vehicle, record the associated remedial action, or
  - where the inspection is not being conducted in conjunction with routine maintenance or rectification of the defect is outside the scope of the RMPC or the capabilities and resources available on the patrol vehicle, nominate the required remedial action
- nominate the need for monitoring based on concerns regarding the visually assessed condition of the structure
- nominate the need for a higher level of inspection based on concerns regarding the visually assessed condition of the structure
- allow the inspector to expand on issues arising from the checklist items in the comment boxes.

The work order number should be recorded where appropriate, permitting the allocation and tracking of inspection expenditure to a particular structure. Details of maintenance activities that are carried out, or are scheduled to be carried out on the structure, should be recorded on the **Structural Maintenance Schedule (A4)**. This identifies the type, nature and cost of any maintenance work carried out and the maintenance problem areas in a particular structure.

It is intended that an entire inspection be carried out within the scope and to the extent specified for this level of inspection and that all the required data fields in the **Routine Inspection Report** form are completed. If a partial inspection is effected, then the inspector must record those items that could not be inspected, together with the reasons for their omission.

In addition to the completion of the **Routine Inspection Report** form, the inspector should photograph and/or sketch any major defects and record the relevant details on the **Photographs and Sketches Record (A1/2)**.

Completed inspection reports are to be entered into the BIS within 30 working days of the inspection, including:

- date of inspection
- name of inspector
- deficiencies flagged by inspector and required actions
- programmed date for an extraordinary Level 2 or Level 3 inspection if nominated
- any limits on the extent of the inspection.

In addition, the inspector shall ascertain whether or not a **Structural Inventory Verification** form was completed for the structure. If so, the inspector shall forward the completed form, along with the inspection report, to the District office within 30 working days of the Level 1 inspection.

Completed samples of standard Level 1 inspection forms for a variety of structure types are included in Appendix A.

2.9 Post-flood inspections

Level 1 inspections are required to be carried out on bridges and culverts following major flood events. For the purposes of this Manual, a major flood event is defined as a flood with an average recurrence interval (ARI) of five years.

Notwithstanding, where a structure is known to have low flood immunity and there is a history of scour damage, it should be inspected following any flooding event where it is overtopped.

2.9.1 Inspection procedure

The following procedures have been developed to provide guidance to inspectors for the post-flood inspection of structures. They have been prepared to determine if the integrity of structures has been compromised by flood water, and so mitigate the risk to road users. Given the geographical scale of the state, remoteness of many structures and scarcity of resources, it is acknowledged that it is often impossible to complete these checks prior to structures being trafficked. In this instance, operational staff will conduct the checks at the earliest opportunity and will be in a position to observe the response of the structure under traffic to assist in the deliberation of structural safety. It is expected that priority will be given to those structures that are known to be susceptible to flood damage.

- Review current drawings, site investigation reports and BIS inventory, risk and inspection records. Review of the latest scour sounding report is essential.
  - Identify any documented alignment discontinuities.

  - Identify the foundation arrangement, scour susceptibility, structure sensitivity to scour and the ‘safe’ scour level. Typical ‘safe’ levels should be taken as:
    - no scour under spread footing structures
    - where the stream bed consists of scour-prone material, there should be at least 4 m of scour-prone material remaining above the pile toe
    - plot data on general arrangement drawing and/or site investigation report and determine ‘safe’ sounding benchmark for reference at the site.
- Check member and structure alignment for discontinuities:
  - kerb alignment (vertical, longitudinal, horizontal)
  - girder alignment
  - bearings/corbels in contact with headstocks and girders
  - restraint angles (movement or damage)
  - expansion joints (closed unevenly, shearing or not level)
  - **Any emergent or increased discontinuities MUST be investigated prior to opening the structure to traffic.**
- Check for scour at supports:
  - Sound for scour holes in accordance with Section 3.11 and compare readings with last inspection results.
  - The creeks will typically be in flood and flowing swiftly; thus, progressively heavier weights and cables/chains may be required to determine the soundings. The use of sonar depth finders may also be considered where flows are too great for weighted chains and conditions allow. In the event that accurate soundings cannot be ascertained due to excessive stream velocities or debris mats, then traffic access should be denied in the case of the scour-prone structures described above. Where there is some confidence concerning the scour performance of the structure then the incremental loading procedure described below could be adopted.
  - **Any emerging scour issues around supports MUST be investigated prior to opening the structure to traffic.**
  - Check upstream and downstream for bank erosion and depositions that may compromise the structure through redirection of flows.
  - **Any irregularities MUST be investigated prior to opening the structure to traffic.**
  - Check structure for debris accumulation. Program the removal of debris from the deck immediately, then the removal of remaining debris once the flooding has receded.
  - **Any irregularities MUST be investigated prior to opening the structure to traffic.**

A checklist template developed by a RoadTek inspector is included in Appendix A5 as a useful prompt for inspecting and recording this information.

- Prior to opening the structure to traffic, the inspector should also check behaviour of structure under vehicular loading. Observe the performance of the structure under increasing vehicular loads (car, 10 T vehicle, 20 T vehicle, and so on) for any anomalous behaviour. Typically the test vehicle will be restricted to the centreline travel on the first pass with subsequent passes in each of the lanes. If the bridge behaves unusually under load, it should be closed until investigated by suitably qualified personnel (that is, Structures Directorate engineers). In this event, a detailed load test will required as specified by BCMAM.

- The inspector should also be looking for the following issues:
  - Large debris deposits in areas that may impede traffic or stream flows, or may affect the performance of the structure (that is, wedged between the end faces of girders at expansion piers). Note that debris deposits in non-critical areas should also be removed, as they may tend to exacerbate the effects of subsequent flooding and may represent a fire hazard.
  - Scour or deposition of material which includes scour of batter protection, embankment material and scour of bed material adjacent to foundations, resulting in undermining of footings. Depositions of material can alter stream flow, which can result in scour of embankments or foundations. Refer to Appendix D for guidance on allowable scour depths for various components.
  - Uplift and movement of superstructure during flood, normally indicated by differential movement in deck elements over piers (that is, kerbs and or deck surface will be noticeably out of alignment at deck joints).
  - Subsidence of foundations which may be a result of scour or loss of bearing capacity of the substrate material. This is indicated by downwards deflection of the deck/kerbs over the deck joint.
  - Debris impact damage to structural elements (for example, girders, piles) which is most likely present in elements on the upstream side of the structure. Extent of the damage can range from mild abrasion and scratching of components to severe structural damage (cracked or spalled/missing concrete, reinforcement deformed or severed, components knocked out of alignment).

2.9.2 Data recording

Where an accredited inspector is undertaking the inspections, they should be completed as a full Level 1 inspection. In addition, a copy of the **Scour Soundings Report (A2/7)** from the previous Level 2 inspection should be obtained and soundings taken. If there is little or no change in the sounding levels from the previous Level 2 inspection (that is, no increase beyond CS 2, or no change in existing CS 3 and 4 readings in accordance with the criteria shown on the form), then the **A2/7** form should be completed, scanned as an image/PDF file and stored in the Photo/Sketches section of the Level 1 report in the BIS. If the inspector has used the inspection checklist template (Appendix A5) then this can also be scanned and saved to the Photo/Sketches section in the same way.

If significant change is observed in the sounding levels, a full Level 2 inspection should be scheduled immediately and a new set of soundings taken during the course of the Level 2 inspection and recorded as per normal.

Where the inspection is undertaken by non-accredited personnel, they should use the template form provided in Appendix A5. Upon completion of the inspection, the details should be forwarded to suitably qualified personnel for review (that is, an accredited Level 2 bridge inspector or engineer) prior to the opening of the structure to traffic. A copy of the completed form should be saved to the Picture section of the Structure Inventory for the structure.

2.10 Post-earthquake inspections

Level 1 inspections are required to be carried out on bridges and culverts following an earthquake.

2.10.1 Inspection procedure

The same general procedures as routinely undertaken following a flood event (refer Section 2.9) should be undertaken following an earthquake. The difficulty associated with post-earthquake response is choosing structures to be inspected for a given magnitude and location.

Following an earthquake, BCMAM will provide guidance to affected Districts of the areas within which structures should be inspected based on primary and secondary influence zones (refer Table 2.10.1) and the following principles:

- Any structure which is located in an area which shows evidence of earthquake damage must be inspected. Generally such evidence will not be visible except for large earthquakes.
- All structures, irrespective of condition state, located within the primary influence zone must be inspected.
- Any structure in condition state 3 or worse, located within the secondary influence zone, must be inspected.

Table 2.10.1 – Critical distances from earthquake hypocentre for various magnitude earthquakes

Earthquake magnitude (Richter scale)	Primary influence zone limiting radius (r_1)	Secondary influence zone limiting radius (r_2)
5.0	4 km	5 km
5.5	10 km	15 km
6.0	13 km	25 km
6.3	30 km	70 km
6.5	36 km	90 km
7.0	46 km	125 km
7.5	123 km	500 km
8.0	158 km	700 km
8.5	185 km	900 km
9.0	500 km	3500 km

2.10.2 Data recording

Where an accredited inspector is undertaking the inspections, they should be completed as a full Level 1 inspection.

Where the inspection is undertaken by non-accredited personnel, the details should be forwarded to suitably qualified personnel for review (that is, an accredited Level 2 bridge inspector or engineer) as soon as practicable. A copy of the completed form should be saved to the Picture section of the Structure Inventory for the structure.

2.11 Post-tsunami inspections

A tsunami event shall be treated in the same manner as a normal flood event (refer Section 2.9).

The checklist template developed for post-flood inspections (included in Appendix A5) should be used to record post-tsunami inspections.

3 Level 2 – Condition Rating Inspection

3.1 Purpose

The purpose of this level of inspection is to rate the current condition of a structure.

This data will be used as a basis for:

- identifying and quantifying structural defects in the structure or its individual components
- identifying and prioritising maintenance needs and/or other actions
- estimating forward budget requirements arising from the maintenance, rehabilitation or replacement needs determined from the condition inspection
- determining the residual life of the structure and appropriate replacement strategy
- re-rating the structure and components after significant maintenance or remedial works have been carried out
- assessing the effectiveness of historic and current maintenance/refurbishment strategies
- assessing the current load carrying capacity
- monitoring the overall condition of the network.

In addition to these, the condition data may also be used as a basis for modelling and forecasting future changes in condition and residual life.

3.2 Scope

The scope of the Level 2 inspection will include:

- Compiling the component inventory (for new structures) and verifying the inventory for refurbished and existing structures. In compiling/verifying the component inventory, the component matrix shown in Appendix B should be referred to. The matrix lists the codes to be used to identify structure components. It also shows the relationship between component groups and components.
- Identifying the exposure classification in the immediate proximity of each component.
- Visually inspecting components to assess their condition using the standard condition rating system outlined in Section 3.8 and Appendix D.
- Reporting the condition of each component and the extent over which that condition applies.
- Rating the overall condition of the structure.
- Identifying structures and/or components which warrant a Level 3 inspection due to a rapid change in condition or deterioration of critical structural components to Condition Rating 4.

- Identifying components which require closer condition monitoring and observation at the next Level 2 inspection because they have deteriorated to Condition Rating 3, show rapid deterioration or demonstrate other features which warrant reporting.
- Capturing a photographic record of the structure and any deficient or non-standard components identified
- Identifying maintenance requirements and/or deficient maintenance practices.
- Identifying supplementary testing as appropriate in accordance with the guidelines of this Manual.
- Verification of the 'Design Inventory' held in the BIS. Prior to the inspection, the inspector shall confirm whether a *Design Inventory Verification form* from BIS has been completed for the structure. If not, the inspector shall forward a completed form, along with the inspection report, to the District office within 30 working days of the Level 2 inspection.
- As Level 2 inspections may require the use of specialist access (for example, Under Bridge Inspection Unit (UBIU), scissor or boom lifts), it is also recommended that, on such occasions, District personnel take advantage of the availability of such equipment and conduct routine maintenance on those components not normally accessible, such as bearings.

3.3 Extent of inspection

3.3.1 General

The Level 2 inspection typically comprises a visual inspection, within 3 m or equivalent (using telescopic equipment), of all components above ground and water level.

The surface of components to be inspected shall be in good natural or artificial light sufficient to observe fine cracks in concrete.

Where present, all bearings at abutments and piers shall be inspected, and bearings from at least one pier shall be inspected at eye level.

3.3.2 Specialist access

As stated in Section 3.3.1, the Level 2 inspection typically comprises a visual inspection to within 3 m or equivalent of all components above ground and water level.

There may be instances where specialist access, in the form of UBIU, scissor or boom lifts, scaffolding and so on, is required in order to access hard-to-reach components to facilitate hands-on inspection. Where specialist access is considered necessary, justification should be forwarded to BCMAM for approval. Structures requiring specialist access are to be agreed between BCMAM and the inspecting authority.

Furthermore, BCMAM is undertaking a desktop review of all structures where specialist access is currently used to confirm each is an effective use of resources.

3.3.3 Additional requirements

In addition to these, the following additional requirements may exist for certain structures:

- Timber bridges: all timber road bridges are subject to a drilling survey (refer Section 3.10).
- Corrugated metal culverts/steel trough decks: ultrasonic testing of steel culvert barrels and steel trough decking on timber girders may be required (refer Section 3.12).

- ACM identification inspection: this is a one-off inspection undertaken on structures with the potential for ACM, as identified in the departmental Bridge Asbestos Register, to visually confirm the presence of potential ACM (refer Section 4.4).
- Underwater inspection: structures designated as requiring underwater inspection shall be inspected at the frequency and to the extent prescribed by the required Level 3 inspection (refer Section 4.5).
- Fracture critical/lack of redundancy: structures meeting the criteria for fracture critical/lack of redundancy shall be inspected at the frequency and to the extent prescribed by the required Level 3 inspection (refer Section 4.6).
- Sub-standard load rating: structures noted as having a sub-standard load rating shall be inspected at the frequency and to the extent prescribed by the required Level 3 inspection (refer Section 4.7).
- Complex/unique structures: structures meeting the criteria for complex/unique structures shall be inspected at the frequency and to the extent prescribed by the required Level 3 inspection (refer Section 4.8).
- Known/suspected deficiencies: structures/structure families with known or suspected deficiencies shall be inspected at the frequency and to the extent prescribed by the required Level 3 inspection (refer Section 4.9).
- Confined spaces inspection: structures (or constituent components) that constitute a confined space hazard such as the interior of box girders (see Section 4.10).

3.3.4 Exclusions

Components that need not be inspected for Level 2 inspections, except as prescribed by Level 3 inspection are:

- interiors of box girders, closed box sections on steel trusses and any other structure/component that constitutes a confined space (see Section 3.3.3)
- areas behind abutments that are inaccessible
- piles and foundations below ground or water level.

These components will be inspected as part of Level 3 inspections where required.

3.3.5 Photography

The Level 2 inspection includes a photographic record of each structure in order to:

- maintain a chronological photographic record of the original structure, any modifications and the waterway
- maintain a chronological record of the structure condition
- provide up-to-date structure images for the Structure Information System.

Level 2 inspection photographic requirements are:

- Inventory record:
  - one general photograph from top of deck showing alignment, width, kerbs and barriers
  - one elevation of structure showing piers, abutments, waterway, cells, headwalls and so on

  - representative photographs of the main superstructure components (for example, girders), from underneath or side of the structure, used in:
    - the original structure
    - any modifications (widening, lengthening, and so on)
  - photograph of any components that do not fall within the defined component classification
- Component condition:
  - all components with a recorded condition rating of 3 or 4 (where a photograph will illustrate the defect)
  - where the same defect occurs more than once and ongoing monitoring is not required, a typical view of the defect may be sufficient
  - close-up photographs of defects should include the component number/location marked on the component in chalk/crayon to allow future identification
  - where a photograph does not provide sufficient detail of a defect, a detailed sketch should be produced which shows the defect and all relevant dimensions.

All photographs must:

- include a digital time stamp
- be taken in natural light (unless defect is in shadow/dark area)
- be composed such that the subject is central and occupies the whole frame
- photographs of defects and non-conforming components must be taken within 3 m of the surface of the component or equivalent using a telephoto lens.
- Inspection frequency

3.3.6 Initial inspection of new and refurbished structures

All new structures and existing structures subject to major maintenance, strengthening or modification (widening, lengthening and so on) shall be subject to:

- Level 2 inspection prior to handover of structure
- Level 2 inspection four months prior to the end of defects liability period.

This is so the Principal Contractor can rectify any defects resulting from construction/maintenance prior to Transport and Main Roads taking responsibility for ongoing maintenance of the structure.

3.3.7 Existing structures

All existing structures shall be subject to Level 2 inspection at the frequencies specified in Table 3.4.2.

Table 3.4.2 – Level 2 inspection frequencies

Structure type	Overall condition state of structure	Inspection frequency (years)
Timber structures and steel culverts (in permanent standing water)	1 – 2	2
	3	1[1]
	4	1[1,2]

Structure type	Overall condition state of structure	Inspection frequency (years)
Bridges and culverts[3]	1 – 2	5
	3	3
	4	1[1,2]
Ultrasonic testing of steel/aluminium culverts without a structural reinforced concrete invert[4]	1 – 2	Not required
	3	Every 2nd Level 2
	4	Every Level 2
Ultrasonic testing of steel trough decks on timber girders[4]	1 – 2	Not required
	3	Every 3rd Level 2
	4	Every Level 2
Tunnels	1 – 2	5
	3	3
	4	1[1,2]
Busway bridges, including elevated and underground stations and pedestrian overbridges at busway stations	1 – 2	5
	3	3
	4	1[1,2]
Other bridges over the road network	1 – 2	5
	3	3
	4	1[1,2]
Retaining structures above/below the road network (excludes retaining structures inspected as part of any other structure)	1 – 2[5]	5
	3	3
	4	1[1,2]
Underwater components (all components other than steel culverts) permanently submerged	1 – 2[5]	8
	3[5]	1
	4[5]	1[1,2]
Confined spaces inspection (all components representing confined space hazards (for example, interior of box culverts))	1–2[6]	8
	3[6]	1
	4[6]	1[1,2]
Large Traffic Management Signs (LTMS) and gantries	1 – 2	2
	3	1[1]
	4	1[1,2]

[1] Level 1 and level 2 inspections to be staggered by six months to ensure inspections occur every six months.

[2] Or at frequency specified by *Structure Maintenance Manual*/Structures Management Plan (whichever is greater).

[3] Includes critical 'minor' culverts.

[4] These conditions only apply where the overall condition state of the structure is attributable to corrosion of the metal barrel / steel trough.

[5] These conditions only apply where the overall condition state of the structure is attributable to the condition of the underwater components.

[6] These conditions only apply where the overall condition state of the structure is attributable to the condition of the components requiring confined space inspection.

Stated inspection frequencies are the nominal time from last inspection subject to a tolerance of two months either way.

3.4 Inspector accreditation

Inspections shall be conducted by trained personnel who also have extensive experience in the inspection, construction, design, maintenance or repair of road structures.

They shall have extensive practical experience and be competent to judge the condition of structures and the importance of visual defects. These inspectors need not be qualified professional bridge engineers, but should have the backing of such a person to aid in decision making or interpreting visual defects or unusual structural action.

Inspectors must attain accreditation through attending a Level 2 training course for bridge inspectors although partial exemption may be granted to suitably experienced inspectors. In addition, it is a requirement that each inspector must undertake a number of inspections and submit these to BCMAM to enable a desktop review to be carried out. The number of inspections required to be submitted is dependent on the structure and material type, details of which are provided in Appendix E. In most cases as part of the assessment process, this will then be backed up by a field audit, in the form of a Level 3 inspection, to ensure compliance with the *Structures Inspection Manual* reporting requirements.

The inspector accreditation appraisal procedure and appraisal forms are included in Appendix E.

3.5 Inspection procedure

3.5.1 Preparation for inspection

Prior to commencing inspections, the inspector shall ensure that all relevant documentation, inspection equipment and safety equipment is in place along with the appropriate arrangements with the relevant road, railway or other Authorities for temporary access as required to carry out the inspection. Safety plans must be prepared and approved.

3.5.2 Data recording

All information obtained from the site inspection shall be recorded on the following forms:

- Condition Inspection Report (A2/1 and A2/2)
- Defective Components Report (A2/3)
- Standard Procedure Exceptions Report (A2/4)
- Timber Drilling Survey Report (A2/5)
- Photographs and Sketches Record (A2/6)
- Scour Soundings Report (A2/7)
- Design Inventory Verification forms from BIS (if required)
- Structural Maintenance Schedule (A4) (if required).

It is intended that each inspection should be carried out to the extent specified for this level of inspection and all relevant data fields in these forms should be completed.

3.5.3 Inspection

At each site, the inspector shall carry out the inspection in a systematic manner starting at carriageway level, proceeding from 'Approach 1' end of the structure down through the superstructure and substructure to the waterway as appropriate.

Approach 1 is defined as the first approach encountered when travelling in the increasing chainage direction.

The inspector shall complete the following activities in accordance with this procedure and the guidelines given in:

Appendix B: Standard Component Schedule

Appendix C: Standard Component Identification Guidelines

Appendix D: Standard Component Condition State Guidelines.

The results of inspection shall be recorded on the appropriate Inspection Report forms from Appendix A.

- Compile (new structures) or verify (for refurbished and existing structures) the inspection component inventory, with attendant exposure classifications, by designated group, component and unique reference number on the **Condition Inspection Report (A2/1 and A2/2)** form.
- Inspect and rate the condition state of each standard component identified and the extent of the component over which the rating applies.
- Assess the overall condition of the bridge and any modification in accordance with Section 3.8.3.3.
- Record separately on the **Defective Components Report (A2/3)** all those components that:
  - require monitoring or further observation at the next programmed inspection, urgent remedial action or a Level 3 inspection
  - are in condition state 4 or have shown a rapid rate of deterioration since the last inspection and require urgent remedial action and/or a Level 3 inspection.
- The inspector is required to give a brief description of the condition and the approximate quantity of the component affected. A photographic and/or sketch record is also required as outlined in Section 3.3.5.
- Record separately on the **Standard Procedure Exceptions Report (A2/4)** form:
  - Components that could not be defined using the standard methodology. A photograph of the non-standard components is also required as outlined in Section 3.3.5.
  - Components that could not be inspected. Reasons must be stated for these omissions.
  - Components where less than 25% is accessible. The exposed portion must still be rated on the **Condition Inspection Report (A2/1 and A2/2)** form.
  - Components which have been made obsolete but remain in place, such as timber girders/corbels that have been replaced by adjacent members. These components shall only be recorded on **Standard Procedure Exceptions Report (A2/4)** form. Any comments relating to the condition of the obsolete component shall be made in the

comments section of this form. Where the inspector believes the condition of the obsolete component presents a risk to the structure or the public this shall be noted in the ‘comments’ section of this form and the ‘overall inspection comments’ field of the **Condition Inspection Report (A2/1)** form. Where the inspector is uncertain if the components are obsolete, guidance shall be sought from BCMAM.

- Sacrificial components (that is, components used to provide support during construction of primary-load carrying components which do not contribute to the load carrying capacity of that component) such as existing sub-standard or defective concrete slab or steel troughing used as permanent formwork when casting a new deck slab. These components shall only be recorded on **Standard Procedure Exceptions Report (A2/4)** form. Where the primary component is obscured by the sacrificial component, the primary component shall be called up as an exception in the **Condition Inspection Report (A2/1)** form. Both the primary and sacrificial component shall be recorded on the **Standard Procedure Exceptions Report (A2/4)** form. Any comments relating to the condition of the sacrificial component shall be made here. Where the inspector believes the condition of the sacrificial component presents a risk to the structure or the public this shall be noted in the comments section of **Standard Procedure Exceptions Report (A2/4)** form and the ‘overall inspection comments’ field of the **Condition Inspection Report (A2/1)** form. Where the inspector is uncertain if the components are sacrificial, guidance shall be sought from BCMAM.
- Any other observation or recommendation not covered by the other forms.

- Record the relevant photographic and sketch details, including reference numbers, locations and descriptions on the **Photographs and Sketches Record (A2/6)** in accordance with the guidelines given in Section 3.3.5.
- Record the results of the timber drilling survey on the **Timber Drilling Survey Report (A2/5)** form where appropriate.
- Record the results of any stream bed profile measurements on the **Scour Soundings Report (A2/7)** form.
- Record the results of any additional inspection requirements (for example, underwater inspection, ACM identification and so on) on the relevant forms.

Completed samples of all standard Level 2 inspection forms for a variety of structure types are included in Appendix A.

3.6 Data transfer

Relevant inspection data shall be entered in the BIS within 30 working days of the inspection. This shall include all photographic records and general descriptive information recorded on the relevant inspection forms, any recommended actions including component inventory amendments, the need for a Level 3 inspection or maintenance action. Photographs shall be saved in JPG file format, and shall be no bigger than one Mb.

Any structurally significant component recorded on the **Defective Components Report (A2/3)** form must be flagged in the BIS as a deficiency, and must remain as such until it has been inspected by a structural engineer and/or rectified.

In addition, the inspector shall ascertain whether or not a **Design Inventory Verification** form from BIS has been completed for the structure. If not, the inspector shall forward a completed form, along with the inspection, to the District office within 30 working days of the inspection.

3.7 Condition rating

3.7.1 General

A fundamental requirement of a systematic inspection procedure, that produces consistent results, is the standardisation and rationalisation of the following variables:

- components that comprise the structure (refer Section 3.8.2)
- condition state descriptions, or level of deterioration pertaining to those components (refer Section 3.8.3)
- classification of the degree of aggressiveness of the environment affecting the rate of deterioration of the component (refer Section 3.8.4).

3.7.2 Compilation of the component inventory

Components and their location shall be designated by:

- modification
- group
- component
- standard component reference

in accordance with the following principles.

3.7.2.1 Modification

Modification refers to the modification status of the structure and is used to associate components with any major modifications undertaken to a structure during its life. Modifications are summarised in Table 3.8.2.1.

Inspection of components that are part of identified widenings are to be assessed and recorded separately to those of the original structure and designated as left or right as viewed from the Approach 1 end of the structure.

Components which are part of other identified modification types are to be assessed and recorded with the original structure, but are to be located with the correct modification classification.

Table 3.8.2.1 – Modification status

Status	Description
O	Original structure
WL_n	Widening to left hand side (when viewed from Approach 1) of structure 'n' denotes number of widening; for example, WL_2 would be the second of two widenings on the left hand side of the original structure
WR_n	Widening to right hand side of structure

Status	Description
L_1	Lengthening of structure at Approach 1 end Note that any abutments which are modified in the course of a lengthening will thereafter be included in the lengthening modification and removed from the original component listing
L_2	Lengthening of structure at Approach 2 end.
S_1	Shortening of structure at Approach 1 end Only components modified by the works shall be assigned the ‘S’ prefix Groups and components removed as part of the works shall be deleted from the inspection inventory
S_2	Shortening of structure at Approach 2 end Only components modified by the works shall be assigned the ‘S’ prefix Groups and components removed as part of the works shall be deleted from the inspection inventory
Re	Re-decking, referring to a change in decking system as opposed to replacement of existing deck components
Ra	Raising Only components modified by the works shall be assigned the ‘Ra’ prefix
St	Strengthening (that is, works undertaken to increase the structural capacity of the structure beyond its original design class) Only components added or modified as part of the works shall be assigned the ‘St’ prefix

Figure B1 and B2 in Appendix B illustrate the concept of modifications.

3.7.2.2 Groups

Principal groups of components comprising approaches (AP), abutments (A), piers (P), spans or culvert cells (S) shall be progressively numbered in the increasing chainage direction of the road.

3.7.2.3 Components

Components as described in Appendix B shall be progressively numbered from left to right as viewed in the increasing chainage direction of the road.

The Component Schedule contained in Appendix B includes a complete listing of standard components along with their corresponding component abbreviations, significance rating, standard reference numbers and units of measurement.

The Standard Component Matrix included in Appendix B shows the association between standard components and groups.

The standard component reference, as defined in Appendix B, must be assigned to the **Condition Inspection Report (A2/1 and A2/2)** form. Standard abbreviations may be used when describing component defects. Where a component observed during the inspection does not conform to one of the predefined components, its details shall be recorded on the **Standard Procedure Exceptions Report (A2/4)** form, the component photographed and the details forwarded to BCMAM for advice.

A series of figures representing the majority of structures likely to be encountered is included in Appendix C to assist with the identification of standard components.

The inspection components are further divided into five material types comprising ‘steel’ (S), ‘precast concrete’ (P), ‘cast insitu concrete’ (C), ‘timber’ (T) and ‘other’ (O). The steel grouping includes aluminium, cast and wrought iron members while the latter comprises brickwork, masonry and any other material not listed.

Precast concrete members can generally be distinguished from cast insitu concrete by the smooth, uniform and dense surface and are typically whiter in colour.

Additionally, when compiling the component inventory for a bridge structure, roadway items such as surfacing, kerbs, joints and bridge railing are typically defined per span. With a culvert structure, this approach is unfeasible due to the significantly shorter span lengths and lack of definitive joints in the deck. For this reason, these components shall be defined per culvert structure and recorded under the Span 1 group, with the corresponding quantities taken from the full length of the structure. Approach items such as guardrail are not affected by this, and are still to be defined separately for both approaches.

In some instances, inspectors may encounter a structure with a configuration that does not fit within the terminology described. Guidance on the designation of groups and components for complex or non-standard structures has been provided in Appendix G, but it is generally recommended that BCMAM be contacted to provide advice on component breakdown of the structure and other related issues.

3.7.3 Condition state criteria

The inspector shall make an assessment of the condition state of:

- each standard component
- the structure as a whole
- any modification.

The condition states have been developed to reflect the discernible stages of deterioration as shown in Table 3.8.3.

Table 3.8.3 – Condition state descriptions

Condition state	Subjective rating	Description
1	GOOD (‘as new’)	Free of defects with little or no deterioration evident
2	FAIR	Free of defects affecting structural performance, integrity and durability Deterioration of a minor nature in the protective coating and/or parent material is evident
3	POOR	Defects affecting the durability/serviceability which may require monitoring and/or remedial action or inspection by a structural engineer Component or element shows marked and advancing deterioration including loss of protective coating and minor loss of section from the parent material is evident Intervention is normally required

Condition state	Subjective rating	Description
4	VERY POOR	Defects affecting the performance and structural integrity which require immediate intervention including an inspection by a structural engineer, if principal components are affected Component or element shows advanced deterioration, loss of section from the parent material, signs of overstressing or evidence that it is acting differently to its intended design mode or function
5	UNSAFE	This state is only intended to apply to the **overall structure rating** Structural integrity is severely compromised and the structure **must** be taken out of service until a structural engineer has inspected the structure and recommended the required remedial action

3.7.3.1 Component condition assessment

The inspector shall make an assessment of the condition of each standard component and the extent over which that condition applies.

In doing so, the inspector shall compare the defects observed in each component with the guidance provided in Appendix D of this Manual for each standard component. These descriptions cannot possibly cover every situation and the inspector is expected to exercise judgement based on his or her knowledge and experience and the guidelines given in Table 3.8.3 to identify the appropriate condition state(s) applicable to each component inspected.

Establishing the mechanism responsible for cracking in concrete elements is crucial to determining the severity of the defect and the corresponding condition of the element. Cracks due to structural and non-structural mechanisms have been differentiated accordingly in Appendix D. If the inspector is unable to determine the mechanism responsible or is not completely confident, then the inspector is to assume the most severe case.

Further guidance on defect identification and causes of deterioration are provided in Part 2 of this Manual.

3.7.3.2 Measurement

The proportion of the component in each condition state shall be determined on the basis of the total visible portion of that component; that is, the portions in each condition state (1, 2, 3 and 4) must add up to the total quantity of that component observed on site. Where there is a single component to be rated (with unit of measurement each), with varying condition state over sections of the component, an estimation can be made of a portion/percentage of area of the component in different condition states. The total value in the ‘quantity field’ must still add up to one.

Each element to be assessed is quantified using one of the following units of measurement:

- number of units making up the element – Each (ea.)
- length of element – Lineal metres (Lin m)
- area of element – Square metres (m2).

The unit of measurement to be used for each of the standard components and associated materials is indicated in Table B-1 of Appendix B.

The percentage of component in each condition state shall be based on the total component that can be observed. Where it is estimated that only 25% or less of the component is visible this fact shall be recorded on the **Standard Procedure Exceptions Report (A2/4)** form, stating the reason why it cannot be fully observed. Such items shall still be assigned a condition state, which shall be based on the visible portion of the component.

In assessing the relative proportions of the component in the various condition states, the inspector should first determine the worst condition affecting the component and its extent then progress through to the best condition pertaining to that component.

A brief description of the defective components shall be recorded in the 'comments' field of the **Condition Inspection Report (A2/1 and A2/2)** form. In addition, the inspector should also indicate the urgency of any required action.

Significant defects found in non-critical structural members which expose the road user to risk and require urgent attention should be noted in the 'comments' field; for example, defective guardrail and connections to the bridge, damaged or defective bridge railing or loose and insecure assembly joints.

Any component which is found to have defects that could compromise the strength or stability of the component, or the structure as a whole, must be rated as condition 4 over the whole of the component. In this event, the defective component must be recorded in the BIS. Immediate remedial action shall be undertaken for this level of defect in all structures.

Typical defects of this nature include:

- fresh scour holes in excess of 4 m deep at piled foundations or any scour below base of spread footing foundations
- flexural cracks in excess of 0.6 mm wide in concrete members
- impact damage to concrete girders which has resulted in exposed reinforcement or prestressing strands
- visible settlement or rotation of substructure elements
- displaced bearings
- pipe rot in timber girders exceeding 70% of the diameter at midspan and/or 50% of the diameter at the supports
- pipe rot exceeding 50% of the diameter of timber piles or corbels
- edge areas of rot in excess of 20% of the cross-sectional area of timber headstocks, or piping rot with a diameter in excess of 90 mm
- snipes in timber girders with a depth exceeding 30% of the diameter of the girder, or snipes in a timber corbel with a depth exceeding 25% of the diameter of the corbel
- 10% loss of section due to corrosion in steel members, fasteners, reinforcement or prestressing tendons at critical sections
- cracking in welds between plates or loss of rivets or bolts (or their effectiveness) in steel connections.

If further advice is required, either a Level 3 inspection shall be commissioned, or sufficient information shall be sent to BCMAM to enable its personnel to conduct a desk-top assessment of the component or structure.

Information to be supplied includes:

- width, extent and location of cracks; for example, 'CW 0.3/L0.3/G1 midspan soffit' denotes a 0.3 mm wide crack, 0.3 m long in the soffit of girder 1 at midspan. Where a number of cracks are present in a single element, this information is best shown in a detailed sketch.
- the area, depth and location of any spalling or loss of concrete cover
- the length and condition of any exposed reinforcement
- residual dimensions of corroded or spalled sections
- lack of connection of guardrail to bridge
- presence of and rate of change of scour depths
- excessive shear deflection or travel on expansion bearings
- magnitude of the forward movement of the top of retaining walls/abutments
- depth of subsidence behind abutments/on approaches
- reference of sketches and/or photographs which detail the magnitude, extent and location of defects.

3.7.3.3 Structure condition assessment

When the inspection of the components has been completed, the inspector shall assess the overall condition of the structure based on observations made at the site in accordance with the condition rating descriptions in Table 3.8.3 and record their assessment(s) on the **Condition Inspection Report (A2/1)** form.

The structure rating shall primarily be based on the condition of the principal structural components such as girders, headstocks, columns, piles and foundations. Principal structural components are those components with a Significance Rating (SR) of 4 specified in Table B-1 of Appendix B.

Except as described in the following, the inspector is expected to exercise judgement, based on his or her knowledge and experience, to determine the appropriate condition state:

- If more than 25% of any principal structural component (Significance Rating 4) in any component group are rated as being in CS 3, then the structure must be given an overall rating of **at least** CS 3. The presence of more serious defects may result in a worse overall rating.
- If more than 25% of any principal structural component (Significance Rating 4) in any component group are rated as being in CS 4, then the structure must be given an overall rating of **at least** CS 4.

Separate ratings for the original structure and any other modifications, comprising widening, lengthening or raising, are required as the construction types and respective conditions are often substantially different.

Where a structure is rated in CS 3 or worse, a summary of the key defects contributing to the assessment should be included in the overall rating comments field on the **Condition Inspection**

Report (A2/1) form. Significant defects found in non-critical structural members which expose the road user and/or general public to risk and require urgent attention should also be noted in the 'comments' field; for example, defective guardrail and connections to the bridge, damaged or defective bridge railing or loose and insecure assembly joints.

Should an inspector consider that CS 5 is the appropriate overall rating of a structure, the inspector **must**, on completion (or partial completion if warranted) of the field work, contact a Senior Inspector at RoadTek Structures Management Services (SMS) to discuss the findings of the inspection. If consensus cannot be reached on the overall condition rating then further advice must be sought from BCMAM. Until such time as SMS has been consulted, and an overall condition rating agreed, the inspection data shall not be finalised in the BIS (that is, marked as 'completed').

3.7.4 Exposure classifications

The exposure classification is a measure of the degree of aggressiveness of the local environment in which the component is situated. If the actual exposure classification is known, as opposed to that assumed in the design, it will assist in assessing the rate of deterioration and/or the residual life of the component or, indeed, the structure.

At the design stage, broad exposure classifications are considered in order to determine and specify the type and quality of materials, protective coating system requirements or amount of cover to the reinforcement and prestressing strands; however, if the quality or integrity of the materials or their protective coatings or cover are compromised, then vulnerable components will become exposed to the local environment. The aggressiveness of that environment will affect the rate of deterioration and, hence, influence the time for repair, rehabilitation or replacement of the component or the structure.

Four exposure classifications approximating those specified for concrete in the Austroads *Bridge Design Code* have been adopted as shown in Table 3.8.4.

Table 3.8.4 – Exposure classification

Exposure classification		Location
Rating	**Environment**	
1	Relatively Benign	Interior of most structures Components above ground on structures located more than 50 km from the coast
2	Mildly Aggressive	Components above ground in structures located between 1 km and 50 km from the coast Components in contact with fresh water or non-aggressive soil[1]
3	Aggressive	Components above ground within 1 km of the coast **not subjected** to direct salt spray Components in very damp environments such as the wet tropics or rainforest areas Any components within 3 m of **permanent** standing water
4	Most Aggressive	Components in tidal or splash zones or those subject to direct salt spray Components in contact with aggressive, contaminated or salt rich soils[1]

[1]The assessment of the aggressiveness of the soil cannot be done accurately without testing but generally can be assumed to be mildly aggressive unless in salt-prone areas, marshes, mangroves, foul smelling soils, landfills or industrial areas. Removal of material around the structure may reveal deterioration indicative of aggressive soils.

3.8 Design inventory data

In almost all cases, the Design Inventory data held in the BIS will have been populated from the drawings when the structure details were first entered into the BIS and, as such, most or all of the fields will have values.

Prior to the inspection, the inspector shall confirm whether the Design Inventory data have been populated and, if so, whether a **Design Inventory Verification** form from BIS has been completed for the structure.

Where the data fields have not been populated in the BIS, the inspector shall notify the relevant regional/District personnel that the data have not been populated in the BIS so that the data can be entered from the drawings prior to the next scheduled Level 2 inspection.

If the data are present but have not been verified, the inspector shall print out a copy of the **Design Inventory Verification** form from BIS and, while on site:

- Compare details on the form with those present on site.
- Where the populated data are correct the inspector shall initial the data (in the corresponding 'initial' box) to confirm data have been verified.
- Where the populated data are incorrect, the inspector shall cross data out and enter the correct information (obtained from the appropriate reference table) in the corresponding box on the form and initial the data (in the corresponding 'initial' box) to confirm data have been verified. If it is not possible to determine or verify the details then the box should be left blank. The reason for not being able to determine or verify the details should be recorded.

A copy of the form is included in Appendix A3.

3.9 Timber drilling survey

Hardwood principal components are subject to additional inspection requirements beyond the scope of a typical Level 2 inspection, the findings of which are recorded on the **Timber Drilling Survey Report (A2/5)** form.

3.9.1 Timber drilling

The purpose of the timber drilling survey is to determine the residual amount of sound timber in a member, normally ascertained by using a drill equipped with a 12 mm diameter bit to bore holes in timber components at critical and suspect locations. The extent and severity of any piping or rot within the component is initially assessed by gauging the resistance to drilling, supplemented by examination of wood shavings. This method relies on the experience and subjective judgement of the inspector and provides information only at the selected drill location.

Test hole locations are probed at subsequent inspections to determine if further deterioration has occurred.

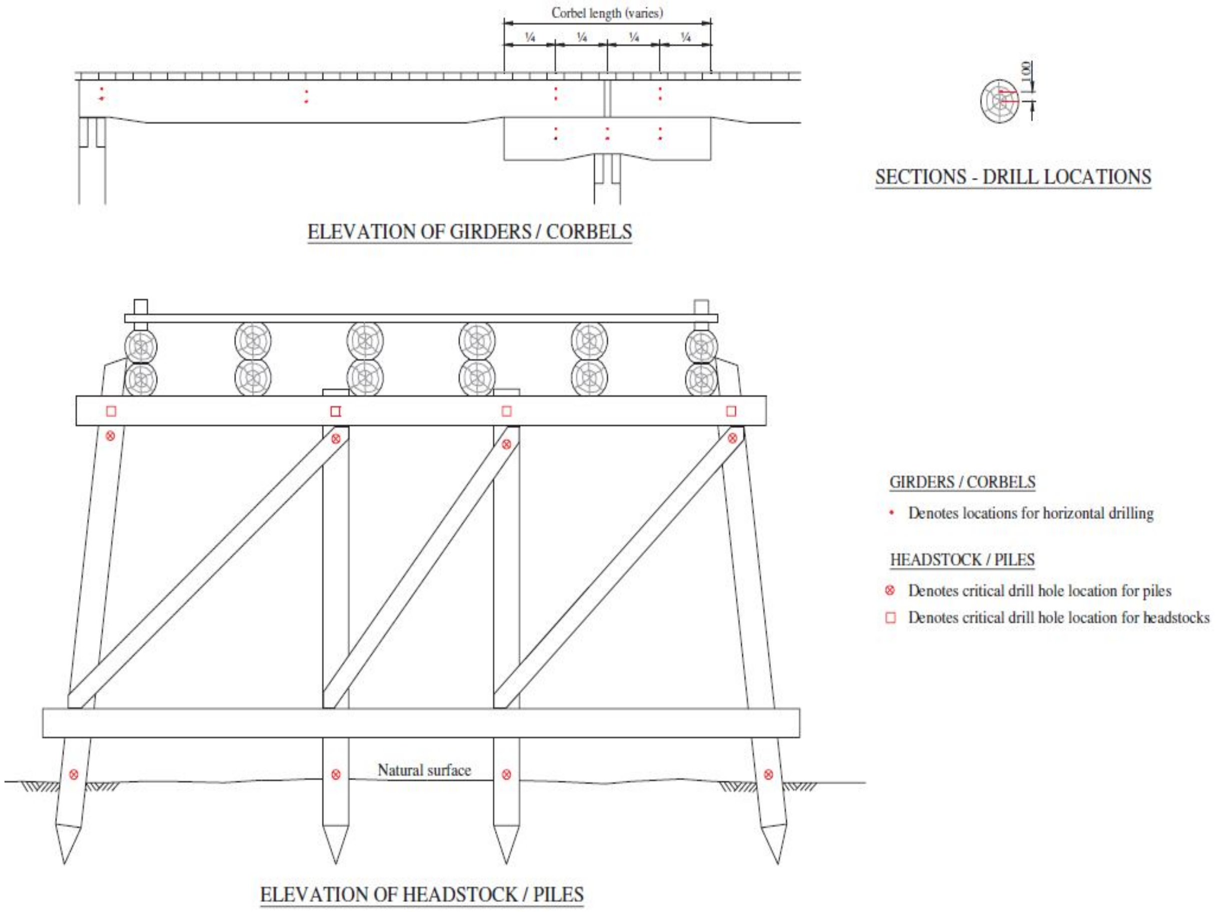
Drilling is carried out at the locations of maximum stress and/or for those areas most susceptible to decay, namely:

- midspan and ends of girders
- ends of corbels
- ends of headstocks

- base and top of end posts
- ground level, normal water level or around connections in piles
- in the upper portion of outer girders beneath the spiking plank
- around bolted connections in general.

Figure 3.10.1 illustrates these critical locations.

Figure 3.10.1 – Timber drilling locations



The following guidelines are provided for the drilling tests:

- At the initial inspection the drilling test holes should be made at the locations illustrated in Figure 3.10.1 and the relevant data recorded on the Timber Drilling Survey Report (A2/5) form.

Note: new timber elements must be drilled at the prescribed locations prior to installation or at the mandatory post-installation inspection.

- All test holes are to be plugged with an approved PVC, coloured plug as follows:
  - assessed condition state CS 1 / CS 2 – blue plug
  - assessed condition state CS 3 / CS 4 – red plug
  - black/yellow plug – not a test hole. Signifies application of pest treatment.

- Care should be taken to avoid drilling completely through members and horizontal drill holes should be inclined slightly upwards to allow drainage.
- Drill holes should be perpendicular to the face of the member so that recorded deterioration is relative to section size.
- Two horizontal test holes are required to be made through the cross-section of girders and corbels (as illustrated in Figure 3.10.1) to detect deterioration in the centre of the member as well as V-shaped deterioration in the top of the member. Note that the component diameter measured is to be the 'width' of the component in the direction of drilling at the point of drilling.
- Additional drill holes are required to be made and probed once the '% consumed' calculation at a mandatory drilling location determines the member to be CS 4. The additional drill surveys shall be carried out at 500 mm intervals along the member until the '% consumed' calculation at a mandatory drilling location determines the member to be CS 2. Table 3.10.1 provides guidance on additional drilling requirements for a girder scenario.

Table 3.10.1 – Additional drilling requirements

Drill results @ standard location			Additional drilling requirements
End 1 (E1)	Midspan (MS)	End 2 (E2)	
Pipe / rot (CS 4)	OK (that is, < CS 4)	OK (that is, < CS 4)	500 mm centres from E1 until CS 2 conditions detected
Pipe / rot (CS 4)	OK (that is, < CS 4)	Pipe / rot (CS 4)	500 mm centres from E1 and E2 until CS 2 conditions detected
Pipe / rot (CS 4)	Pipe / rot (CS 4)	Pipe / rot (CS 4)	At ¼ points of the component
Pipe / rot (CS 4)	Pipe / rot (CS 4)	OK (that is, < CS 4)	At ¼ points between E1 and MS and 500 mm centres from MS – E2 until CS 2 conditions detected

For each test location, the inspector is required to capture the component location ('Modification', 'Group', 'Component' and 'Standard Number' as per the **Condition Inspection Report (A2/1 and A2/2)** form along with the 'Component Diameter'). In addition the test details and test results are recorded as follows:

Test details:

- Location: this is the position of the drill hole on the component being drilled. Guidance is provided on the **Timber Drilling Survey Report (A2/5)** form.
- Diameter: this is the diameter of the hole drilled.
- Orientation: orientation of the test hole (Horizontal (H), Vertical (V) or Other (O)).

Test results:

- Solid: diameter of solid remaining timber.
- Rot: depth of rot detected.
- Pipe: diameter of piping decay detected.

Using these, the % of component diameter consumed is calculated by the BIS which, for piles, girders and corbels, can be used to determine the condition state. Guidance for calculation of condition state is provided for each component type in Appendix D.

3.9.2 Snipe depth measurement

In addition to drilling of timber components, the inspector is required to measure the depth of snipes located at the ends of girders and/or corbels and record the measured value in the ‘Snipe Depth’ column.

The allowable snipe depth ranges outlined in Appendix D are used to determine the appropriate condition state due to the snipe depth.

When horizontal cracks propagating from the root of the snipe are present, the length of crack must be recorded in the ‘comments’ field on the **Timber Bridge Drilling Survey Report (A2/5)**. In addition, the presence of anti-splitting bolts (or any other treatment to limit splitting) installed at the girder end must also be recorded.

3.9.3 Undersize girders

Where the measured girder diameter is less than that shown on the Standard Drawings by more than 20 mm, then the ‘undersize’ check box should be ticked and the required girder diameter recorded in the ‘comments’ field.

In addition, undersized girders should also be noted in the ‘overall inspection comments’ field on the **Condition Inspection Report (A2/1)** form.

3.9.4 Timber member condition rating

For each test location, the inspector shall record the worst condition state (due to drill results, measured snipe depth or girder size).

Where the worst condition state recorded for each component is more critical than the observed condition state (from the visual inspection), this shall be recorded on the **Condition Inspection Report (A2/1)** form.

3.9.5 Alternatives to drilling

A number of alternative ‘non-destructive’ technologies have been trialled by the department to determine if more accurate testing results can be obtained. These technologies are:

- ground penetrating radar (non-destructive)
- nuclear densometer (non-destructive)
- resistograph (quasi-destructive).

The radar method records reflected radar signals to produce a continuous readout over the length of a member. Interpretation of results is difficult but it is claimed that calibrated tests on a number of decommissioned girders have proven that an experienced operator can identify the extent and location of internal defects. This method is not considered suitable, given the expert interpretation required.

The second method uses an isotope source and detector which also gives a continuous readout along a member. This method returns a measure of soundness by determining the average density along a member. Site trials on decommissioned girders were encouraging although some limitations were identified in relation to calibration, requirement for mounting on a centralising jig/bracket and the need to undertake additional testing to verify results.

The resistograph method uses a 2 mm diameter drill bit to drill the member at a constant feed rate and rotational speed. The unit measures resistance encountered as the bit advances into the timber to detect decay, voids and other irregularities in the member. When these trials were originally carried out, measurements were output to a paper trace. The current resistograph units' have on-board storage as well as a real-time electronic display of results. The trials indicated that the resistograph method provides an accurate portrayal of internal soundness but, as for conventional drilling, only provides results at discrete drilling points.

A full report on the trials is included in Appendix D of the *Timber Bridge Maintenance Manual*. This includes recommendations for the appropriate use of these technologies to supplement conventional drilling.

3.10 Scour survey

Some types of scour, such as that caused by gradual degradation of the stream bed over a period of years or a number of flood events, can be difficult to identify due to factors such as vegetation regrowth.

The reliable and consistent checking of the waterway profile for channel degradation, aggradation and localised scour progression over time can only be made by a measurement of the stream bed level from a permanent local reference point.

The following process, referred to as 'sounding', shall be adopted as an integral part of a Level 2 inspection. Figure 3.11 illustrates the application of this process.

- If the stream bed is exposed, then the sounding height from the top of the kerb or other convenient permanent feature (such as the top of a concrete parapet) is to be measured down to the stream bed at midspan and at either end of each span on the downstream and upstream sides of the structure.
- If there is standing water at a bridge site, then the sounding height from the kerb or other permanent reference feature on the superstructure is to be measured down to the water surface, and then down to the stream bed at midspan and at either end of each span on the downstream and upstream sides of the structure.
- Where localised scour holes are identified, the inspector shall take stream bed measurements at 1.0 m intervals in the vicinity of the area in which the local scour was identified. Measurements shall be taken until the extent of the localised scour has been determined. A 'localised scour' will satisfy the following criteria:
  - A significant change in scour depth is noted at a particular measurement location. This may be at a midspan location, an abutment or at a particular pier (in which case, both of the adjacent span ends may have been affected).
  - Both the upstream and downstream points for the particular location may have been affected.
  - The 'change' tapers off rapidly, to the extent that soundings taken at adjacent locations show little or no change from previous/first readings.

  – For example, if referring to Figure 3.11, both the existing 'localised scour' and the added scour hole shown in red would be considered as 'localised scour'. As such, they will be rated in accordance with the local scour criteria. All other soundings will be rated in accordance with the 'change in depth' criteria.

- At abutment spillthroughs, measurements shall be taken at the top of the abutment protection (to act as a known ‘reference’ point for investigation of the spillthrough – shown as ‘O’ in Figure 3.11) and approximately 500 mm beyond the point where the waterway bed meets the spillthrough (the second ‘reference’ point). The approximate measurement locations and anticipated distances should be determined from design drawings and, where possible, the initial reference readings should be taken in ‘dry’ conditions so that distances may be confirmed visually. During or immediately after flood events, the abutment spillway shall be dipped at a few locations (between (and including) the first and second reference points). If the dipping reveals that the abutment protection is exposed to a depth significantly greater than previously recorded, the full set of measurements are to be forwarded to the Director (BCMAM) for further advice.

After the first round of ‘soundings’ has been completed, inspectors shall ensure that they document stream bed ‘sounding’ depths from the previous inspection for comparison with readings obtained during the current inspection.

The recorded condition state for each scour sounding is the highest condition state determined from the difference between the current reading and the previous inspection or the first (as-constructed) sounding.

In the event that reconstruction/rehabilitation of the abutment protection or channel bed is undertaken, the new bed levels shall be captured (as part of the required Level 2 inspection following refurbishment). Details of any such bed modification shall be recorded as follows:

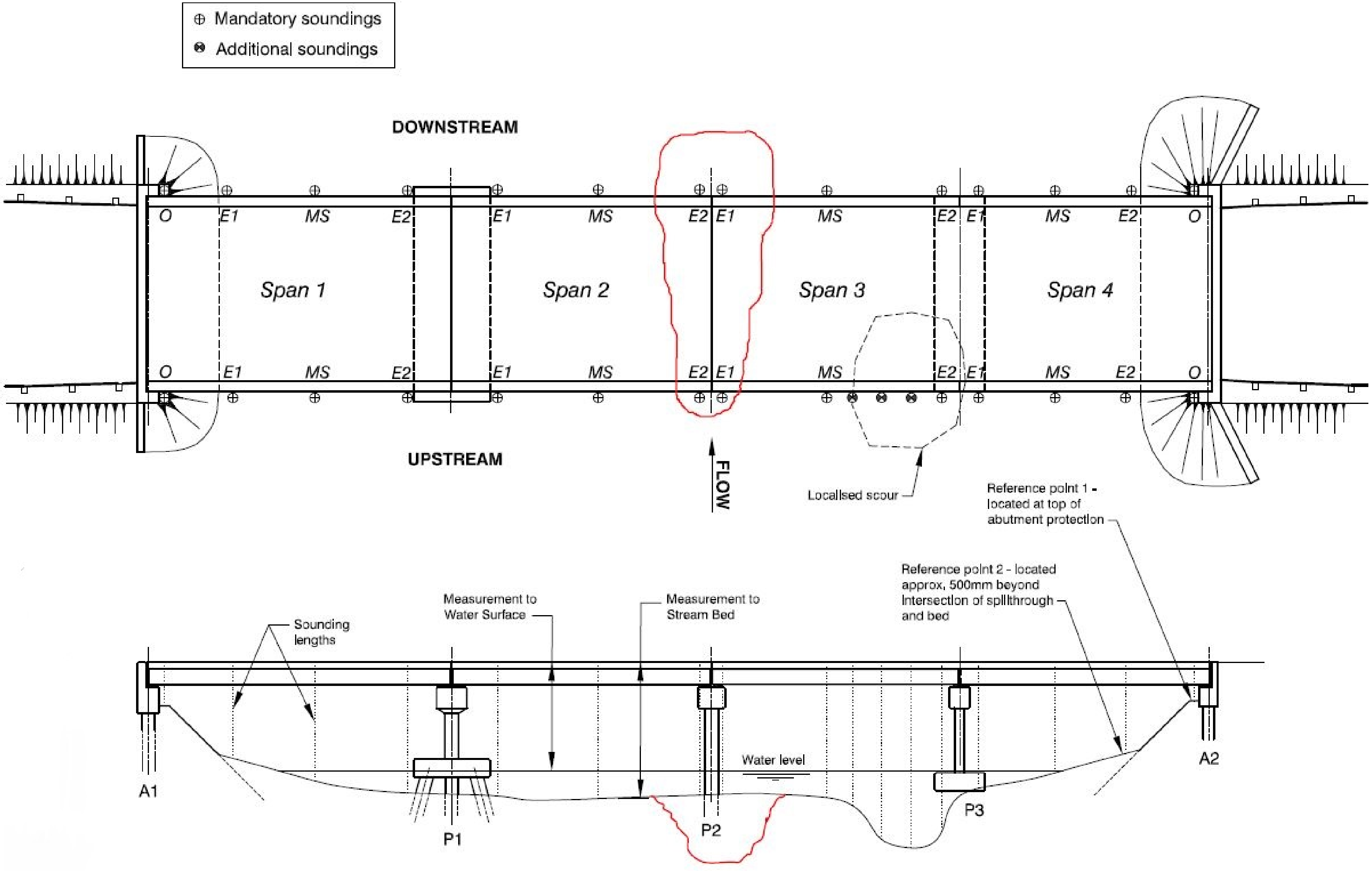
- Tick the ‘Rehab’ check box for each measurement indicating reconstruction/rehabilitation has been performed at the measurement point.
- Once the ‘Rehab’ check box is ticked, the ‘First’ and ‘Previous’ measurements are removed and the condition state reset to CS 1.
- The sounding taken as part of the post-rehabilitation Level 2 inspection is used as the ‘First’ reading for all subsequent inspections.

Measurements shall be taken using a standard measuring tape with a small weight (1 – 2 kg approx.) fastened to the end. Sonar depth-finders have been used successfully in post-flood conditions (fast flowing water), with the transducer attached to a rigid pole and submerged, where standard ‘dipping’ with a weighted tape is not possible. As these units operate on sonar, they are, however, susceptible to the presence of debris or heavily silted water so consideration must be given to the conditions on site at time of inspection.

Results of the ‘soundings’ shall be recorded on the **Scour Soundings Report (A2/7)**.

The locations from which the ‘soundings’ are measured shall be recorded on the form, with the precise locations marked discretely on the structure using epoxy or other suitable material (experience has shown that paint/pen marks tend to be erased when submerged). Inspectors shall endeavour to take measurements from the same locations at future inspections.

Figure 3.11 – Scour soundings



3.11 Ultrasonic testing

The department manages a large number of thin-walled buried corrugated metal culverts which are considered high-risk structures due, in part, to the potential for corrosion along the top of the wetted perimeter or in the invert. Corrosion of these culverts is expected in-service and a large proportion of the wall thickness is provided to both assist stability of the culvert barrel during construction and to act as a sacrificial component; however, if left unattended, corrosion of the culvert walls can result in longitudinal perforations in the culvert and loss of structural integrity. Furthermore, compacted backfill surrounding the culvert can also be lost through erosion, destabilising the culvert.

In addition to these culverts, the department also manages a number of timber bridges with steel trough decking systems. This decking system is also susceptible to corrosion and, while this may not constitute as high a risk as in buried corrugated metal culverts, corrosion usually occurs on the non-visible face, making it difficult to detect corrosion.

Inspectors are, therefore, required to actively monitor the residual thickness of culvert walls and steel trough decking as part of the Level 2 inspection at the frequencies specified in Table 3.4.2. Traditionally, this was undertaken by drilling small diameter holes and measuring remaining thickness. Non-destructive ultrasonic testing (UT) is now the preferred approach.

UT should only be undertaken by operators trained and familiar in the use of the UT equipment being employed. Furthermore, it is essential that the correct probe is employed to ensure accurate results. The following is provided to assist with equipment selection:

- The probe needs to be a small diameter twin crystal of high frequency. As the thickness of the culvert walls and deck troughing can be up to 10 mm, a probe with a 10 MHz frequency or higher is required.
- To calibrate the probe, a step-block is required. A step-block is a certified piece of metal containing ‘steps’ of various thickness. The use of this calibration tool eliminates the need for drilling proof holes in the structure. For testing of the structures previously mentioned, a block containing steps of 1 to 10 mm in thickness would be suitable.

The process following provides some guidance as to how to carry out UT:

- Before arriving on site, prepare sketches and/or tables to record the locations of the scans and results. Refer to Appendix A6 for a template suitable for both structures and guidance on test locations.
- To assist with monitoring the deterioration of the structure, it is recommended that comprehensive records of the scan locations and recordings are kept and attached in the BIS with the inspection. Additionally, reference points on the structures should be marked so future scans can be carried out at the same locations.
- Using a grinder, remove any protective coating (for example, galvanising) from at least two areas of sound material. If no coating is present, then remove any residue to expose a clean surface. For aluminium culverts, care should be taken to avoid damaging the protective oxide coating.
- Apply couplant to the cleaned areas, scan and record the readings.
- Identify corroded areas requiring scanning and repeat the process outlined previously.

- Following scanning, touch up cleaned areas using a cold galvanising paint included in the list of approved products.

Note: no zinc-rich compounds are to be used to repair aluminium culverts.

To ensure the level of deterioration of a culvert base is accurately recorded, it is important to carry out the scanning when no water is present. This can be achieved by either timing the inspection to coincide with dry conditions, locally diverting the water within the test barrel, diverting flow into another cell or damming the inlet and dewatering.

Where diversionary/dewatering or access requirements are significant, careful consideration shall be given to the benefits and frequency of UT testing. This needs to take into account such factors as age, environment, observed condition and the like.

3.11.1 Culverts with invert linings

For culverts containing structural invert linings (that is, the lining renders the underlying metal culvert redundant), no testing is required.

For culverts with an AC or shotcrete lining, it is necessary to test the culvert invert at discrete locations using an approved method to remove the lining without risk of damage to the underlying metal wall.

Test locations

The following locations in each pipe will be tested (refer to Appendix A6 for further details):

- Testing shall be undertaken at the following locations:
  - in the middle of the pipe (this location is MANDATORY)
  - at points 1 – 2 m in from either end of the pipe; if the pipe is short enough so that coming in 2 m from the end will bring you within 2 m of the middle core location, then these end cores are not required
  - for pipes with a length greater than 15 m, it may be necessary to take a single set of intermediate cores between the middle and end locations; this will be dependent on the condition of the exposed portions of the pipe at the other core locations
  - if test results are consistent between middle and end cores, then additional intermediate cores will not be required
  - significant variation in results between middle and end cores may require additional intermediate cores to be taken; this decision will be made at the discretion of the inspector.
- At each of these locations, cores shall be extracted:
  - at the culvert invert (Test Point 1)
  - approximately 200 – 300 mm below the top of the invert lining (Test Points 2 and 3).

3.11.2 Other issues

- Evidence of scour and water flows under the invert should be investigated and reported. Any voids should be backfilled as soon as practicable and an insitu concrete invert formed over the corroded/missing section.

- Any structural deformation or evidence of deformation of the road surface must be regarded as an indication of failure and should be investigated by an engineer immediately and referred to BCMAM for advice. Inspectors should ensure that the road surface immediately above the pipe is checked for deformation, thus special care should be taken to ensure the alignment of skewed culverts is correctly determined using ranging poles or similar.
- In the event of surface deformation, particularly following wet weather and/or high water flows, then propping or road closure should be considered.
- If the culvert contains permanent standing water due to deformation of the invert or poor design, then the structure should be regarded as very high risk and a Structure Management Plan should be developed to manage it. In all cases, the presence of permanent standing water should be recorded during inspections and reported to District maintenance personnel, and maintenance activities programmed to ensure that water can flow freely through the culvert (that is, clear inlets and outlets). Particular attention should be paid in coastal agricultural areas, where the chemical composition of the run-off water (that is, sulphates) is likely to exacerbate the deterioration of the culvert.

4 Level 3 – Special Inspection

4.1 Introduction

Level 3 inspections are detailed engineering investigations that generally include a combination of field investigation and theoretical analysis. They generally target or address a specific issue relevant to an individual structure or a class of structures.

Level 3 inspections are intended to provide improved knowledge of the condition, load carrying capacity, in-service performance and other characteristics that are beyond the scope of Level 1 and Level 2 inspections.

4.2 Detailed structural engineering inspection

4.2.1 Purpose

This is an extensive inspection carried out by a structural engineer, which may include physical testing and structural analysis in order to:

- assess the structural condition and behaviour of a structure
- identify and quantify the current and projected deterioration
- develop appropriate management strategies.

4.2.2 Scope

The scope of the detailed structural engineering inspection will include:

- review of any previous inspection and testing reports
- review of traffic counts, traffic studies, Culway or WIM records and planning reports which include the structure
- review of environmental factors (REFs) including contaminated site records
- determination and programming of equipment and resources required for the inspection (in conjunction with the District) including preparation of a safety plan

- detailed inspection of all relevant bridge components including such measurements, testing and analyses as necessary to supplement the visual inspection
- determination of material properties and structural behaviour
- identification of components which are limiting the performance of the structure due to their current condition and capacity or are likely to deteriorate to such a level within the next five years
- identification of the probable causes and projected rate of deterioration and the effects of continued deterioration on the performance, durability and residual life of the structure
- identification of factors which will influence the dynamic load allowance to be used in load rating; these factors include the geometry and quality of the bridge approaches, surface discontinuities at deck joints and the dynamic response of the bridge
- examination of the hydraulic performance of the structure including any signs of siltation, scour, debris impact or build-up, bank or embankment erosion and tree and vegetation encroachment.

4.2.3 Inspector accreditation

Detailed structural engineering inspections shall be carried out by or under the supervision of an experienced Registered Professional Engineer of Queensland (RPEQ) bridge engineer. Inspections must be arranged through the Director (BCMAM) of the department's Structures Directorate. Structures Directorate is the preferred supplier of inspection services.

4.2.4 Inspection frequency

Unlike Level 1 and Level 2 inspections that are undertaken at predetermined frequencies, a Level 3 inspection or investigation is undertaken on an as-needs basis.

A detailed engineering inspection will be carried out in one of the following circumstances:

- in order to assess the condition of a structure prior to carrying out programmed works such as rehabilitation, strengthening or widening
- as the result of recommendations in a Level 2 Bridge **Condition Inspection Report (A2/1 and A2/2)** form which has rated the structure condition as poor or a principal component in condition state 3 or 4
- to provide a load rating for the structure
- to examine the difference between a theoretically structural deficient bridge to determine if the bridge exhibits distress compatible with the calculations; in some circumstances, bridges may be grouped in families of similar structures
- to prepare a Structures Management Plan (SMP1) or other reports.

4.2.5 Extent of inspections

A project-specific brief for the detailed structural engineering inspection shall be prepared by the District Director's delegate in consultation with a Principal Engineer from Structures Directorate.

4.2.5.1 Field inspection

As a minimum, the field inspection component will comprise a visual examination of all readily accessible components of the structure, supplemented, where necessary, by examinations, testing or analyses specified in the project-specific brief such as:

- underwater inspection of submerged components
- geotechnical investigation including drilling, instrumentation and monitoring
- hydraulic investigation of dynamic flood effects including assessments of flood forces, scour sedimentation, debris size, formation and impact
- location of reinforcement using cover meter
- coring and testing of concrete to assess strength and durability parameters including compressive strength, density, aggregate reactivity and depth of penetration of carbonation and chlorides
- measurement of half-cell potential and resistivity of reinforced concrete components
- examination of steel members using methods such as dye penetrant, magnetic particle, radiographic, ultrasonic or x-ray
- measurement of corroded member dimensions
- static or dynamic load testing of the structure.

4.2.5.2 Structural analysis

A load capacity assessment may be included in the brief to determine the repeated live load capacity for the remaining service life of the structure. The assessment shall be based on:

- original design drawings and specifications
- ‘as-built’ construction records (including pile driving, material testing, modifications, amendments and defect records)
- material properties, workmanship, condition and loading determined by field inspections, tests and direct measurement.

4.2.6 Data recording in the field

Data recording requirements will be in accordance with those specified by the project-specific brief.

As a minimum, data recording will be similar to that required for a Level 2 inspection with additional references to record the identification references, types and locations of all testing and sampling conducted as part of the inspection.

Component designation and condition rating shall be in accordance with the requirements of Section 3.

4.2.7 Reporting

A written report shall be submitted to the District Director, with a copy to the Principal Engineer (BCMAM) of Structures Directorate, within 60 days of the inspection and shall include the specific requirements outlined in the project brief.

The District Director shall consider the recommendations of the report and generally shall initiate the necessary actions. If the District Director does not agree with the recommendations, a response to that effect shall be given in writing to the inspecting engineer and copied to the Principal Engineer (BCMAM) within 30 days of receipt of the inspection report.

A copy of the final report shall be forwarded to the Principal Engineer (BCMAM) who shall be responsible for entering the following salient details into the BIS within 30 days of completion of the report:

- an executive summary of the written report, including the distribution list
- a summary of all other reports produced in order to supplement the Level 3 inspection, such as diving surveys and materials testing
- rating of all primary defects, identification of deterioration mechanisms and determination of the overall condition of the structure
- results of any load capacity assessment conducted (desktop assessment, or static and/or dynamic load testing)
- Bridge Equivalence Ratings (if required in accordance with the brief).

A record of the photographs are to be included in the written report. Photograph size and quality is to be in accordance with Section 3.7.

If, on completion of the detailed engineering inspection, the condition rating of the component(s) of concern and/or the overall rating condition of the structure are found to differ from the Level 2 report that generated the request for further detailed inspection, then a new Level 2 inspection report shall be produced as follows:

- Generate a new inspection report with revised condition ratings for the components inspected.
- All other components shall be recorded with the same condition ratings as noted at the last Level 2 inspection.
- Date of inspection shall be noted as the date of the detailed engineering inspection.
- A statement similar to the following shall be recorded in the ‘overall inspection comments’ field.

  This inspection report has been generated to reflect revised condition ratings for the [Insert component name here] *which were subject to a detailed engineering inspection on the above date. No other components other than* [Insert component name here] *were inspected and condition ratings for all other components are as recorded on* [Insert date of last Level 2 inspection here], *which is the last ‘full’ Level 2 inspection.*

- The date of the next inspection shall be recorded as the appropriate frequency from the previous ‘full’ Level 2 inspection; that is, if the next inspection, based on revised overall condition rating is three years then the next inspection is to be scheduled for last full inspection date + three years.

4.3 Asbestos containing material identification inspection

4.3.1 Background

The hazards associated with exposure to airborne asbestos fibres are well documented and there are numerous documents available relating to the management of asbestos. To effectively mitigate any risks associated with potential asbestos exposure when inspecting/working on highway structures, Transport and Main Roads has prepared MRTS96 *Management and Removal of Asbestos* which outlines the roles, responsibilities and necessary steps required when working in the presence of ACM.

In accordance with legislative requirements to prepare and maintain an asbestos register, the BIS has been amended to register ACM in structures and is being maintained up-to-date by BCMAM.

Asbestos may be present in highway structures constructed before 2003. Possible locations of ACM include:

- permanent/sacrificial formwork between girders on concrete and steel girder constructed bridges
- deck unit constructed bridges with cast insitu bridge decks (that is, no post-tensioning)
- external and internal service mains pipelines such as stormwater and sewer pipes
- service pits
- internal service conduits
- half pipes for drainage channels
- drainage systems on bridges or in close proximity to bridge
- asbestos bonded buried corrugated metal culverts.

It is almost certain that any compressed fibre products used on structures constructed and completed prior to 1985 will contain asbestos. The probability that asbestos is impregnated in compressed fibre products declines from 1986 – 2003. ACM is unlikely to be present in the following:

- transversely stressed precast deck unit superstructures
- concrete box girder superstructures.

Note: The presence of ACM does not in itself represent a hazard to the safety of employees or the community at large. Inhalation of airborne fibres represents the safety concern and, hence, it is only when ACM is disturbed either accidentally (for example, vehicle impact) or intentionally through activities such as strengthening, refurbishment or demolition that the ACM becomes a hazard. No disturbance can reasonably be expected to occur through routine maintenance activities or inspections (Level 1 and Level 2).

4.3.2 Purpose

The role of BCMAM, through the Program Manager, Statewide Structures Management Project is to facilitate the identification of structures with the potential for ACM.

The purpose is to identify the potential permanent/sacrificial inclusion of asbestos on departmental structures. These inspections will be conducted to ensure that the department's asbestos register is up-to-date with the Work Health and Safety counterpart.

4.3.3 Scope

For structures other than those where there are reasonable grounds to believe asbestos is not present, the Program Manager will make arrangements for a visual inspection by an experienced Level 2 Transport and Main Roads bridge inspector.

This is a one-off inspection undertaken on structures with the potential for ACM, as identified in the department's bridge asbestos register, to visually confirm the presence of potential ACM. Under no circumstances shall suspected ACM be disturbed during the inspection.

The inspection may be undertaken as part of programmed Level 1 or Level 2 inspections, subject to the inspector being made aware of this requirement prior to the inspection.

This inspection will not involve hands-on practices (that is, using Under Bridge Inspection Unit (UBIU) or similar equipment).

4.3.4 Procedures and inspector accreditation

ACM identification inspections shall be carried out in accordance with these procedures.

The outline procedure to be followed is illustrated in Figure 4.3.6a.

ACM identification inspections must be completed by a Transport and Main Roads-accredited Level 2 inspector.

4.3.5 Frequency

The inspection will be undertaken once only, at the earliest available opportunity, as part of the Level 1 (subject to experience/accreditation of person undertaking the inspection) or Level 2 inspection program.

4.3.6 Data recording

It is essential that the existence of a potential asbestos containing product at a bridge be identified to ensure that departmental staff or contractors and consultants who may be engaged to work on the bridge are aware of the hazard. The results and data obtained by the inspection shall be recorded in the BIS asbestos register managed by BCMAM.

On completion of the ACM identification inspection, the following actions shall be completed:

- If no elements, that may contain asbestos, are identified:
  - the asbestos register shall be updated with the following statement:

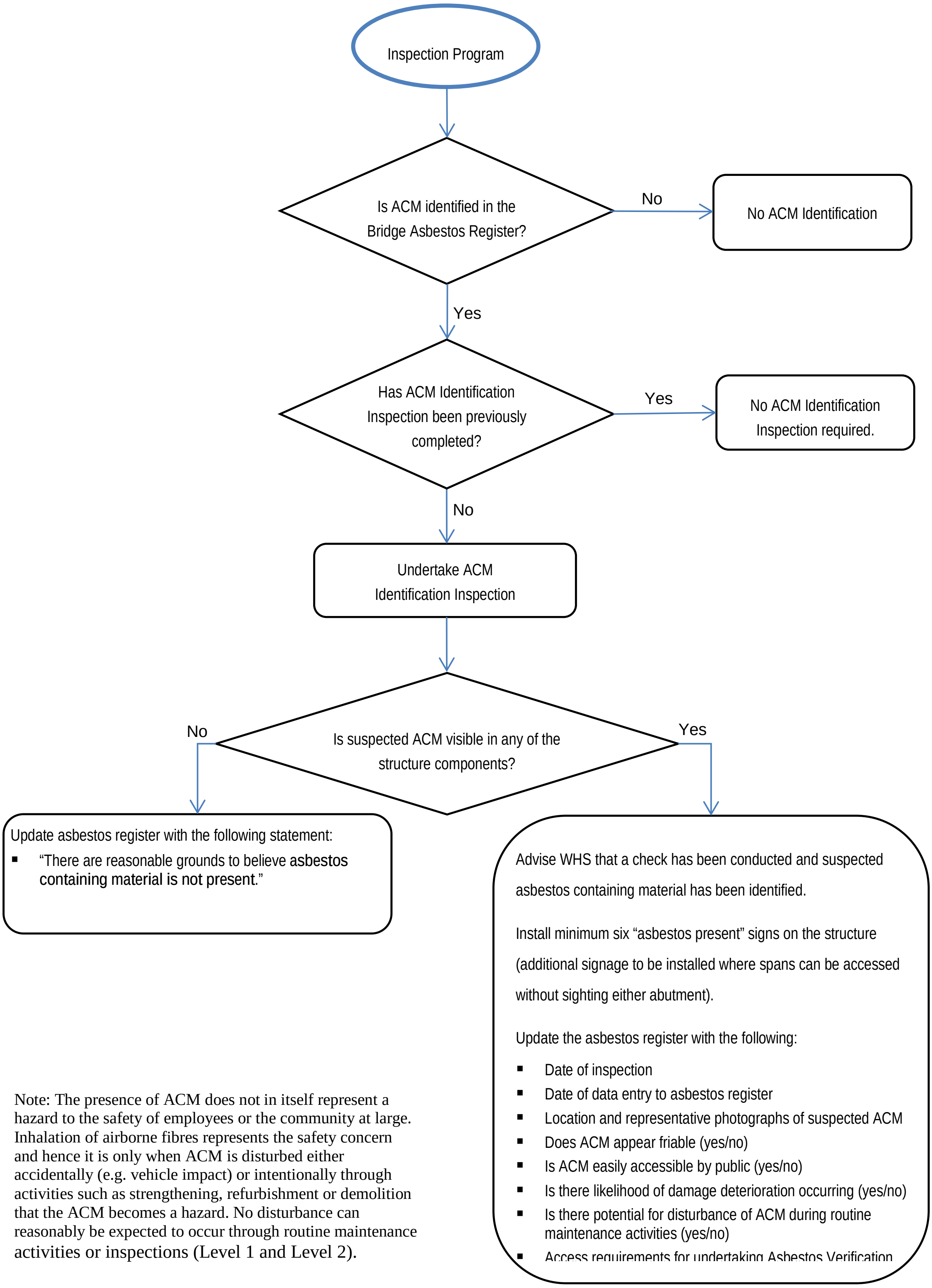
    There are reasonable grounds to believe asbestos containing material is not present.

- If elements are identified that may contain asbestos:
  - advise Workplace Health and Safety that a check has been conducted and potential asbestos containing material has been identified
  - place a minimum of six 'Asbestos Present' signs on the structure (refer Figure 4.3.6b), one sign to be located at each corner of the structure and one on each abutment (or equivalent) clearly visible from beneath the structure; where access to any span of a multi-span structure is feasible without sighting either abutment, then signage shall also be erected on the face of each pier facing the accessible span

- update the asbestos register with the following information captured during the inspection:
  - date of inspection
  - date of data entry to asbestos register
  - location and representative photographs of elements suspected of containing asbestos
  - approximate quantity of ACM and unit of measurement (m^2, m, number)
  - does element appear friable? (yes/no answer based on visual assessment only)
  - is ACM easily accessible by public? (yes/no answer)
  - is there a likelihood of damage or deterioration occurring? (yes/no answer based on judgement)
  - is there potential for disturbance of material during routine maintenance activities? (yes/no answer)
  - access requirements for undertaking asbestos verification inspection
  - exceptions report identifying areas of the structure with potential ACM that were not inspected
  - is the presence of asbestos able to be confirmed based on the visual inspection only? (for example, testing already undertaken on similar components on identical structures constructed on same length of highway under the same contract).

- In addition, the asbestos register shall be updated with the date that 'Asbestos Present' warning signs were installed on site.

Figure 4.3.6a – Procedure for asbestos containing material identification inspection



Note: The presence of ACM does not in itself represent a hazard to the safety of employees or the community at large. Inhalation of airborne fibres represents the safety concern and hence it is only when ACM is disturbed either accidentally (e.g. vehicle impact) or intentionally through activities such as strengthening, refurbishment or demolition that the ACM becomes a hazard. No disturbance can reasonably be expected to occur through routine maintenance activities or inspections (Level 1 and Level 2).

Figure 4.3.6b – Asbestos hazard warning sign

4.4 Asbestos verification inspection

4.4.1 Purpose

This requires the Program Manager, Statewide Structures Management Project to engage an accredited National Association of Testing Authorities (NATA) inspector to conduct tests under ISO 17020.The purpose of the asbestos verification inspection is to confirm the presence of asbestos in suspected ACM where the material may be disturbed through any proposed activity on the structure.

This procedure is in accordance with the relevant Acts, Regulations and Codes of Practice.

4.4.2 Scope

This inspection will involve hands-on practices (that is, using under bridge inspection unit (UBIU) or similar equipment) to gain access to the areas of concern and may involve the breaking back of limited areas of concrete to facilitate removal of samples for testing by a NATA-accredited laboratory under ISO 17020.

4.4.3 Procedures and inspector accreditation

Asbestos verification inspections shall be carried out on any structure with suspected ACM (as noted in the asbestos register) where proposed activities may result in disturbance of the suspected ACM.

Inspections must be undertaken in accordance with these procedures.

The outline procedure to be followed is illustrated in Figure 4.4.5.

Asbestos verification inspections must be undertaken by a licensed asbestos assessor and testing must be undertaken by a NATA-accredited laboratory.

4.4.4 Frequency

The inspection will be undertaken once only, during the planning stage for the activities that may result in the disturbance of the suspected ACM.

4.4.5 Data recording

It is essential that presence of asbestos be identified to ensure that departmental staff or contractors and consultants who may be engaged to work on the bridge are aware of the hazard and that appropriate control measures can be implemented.

The results and data obtained by the inspection shall be recorded in the BIS asbestos register managed by BCMAM.

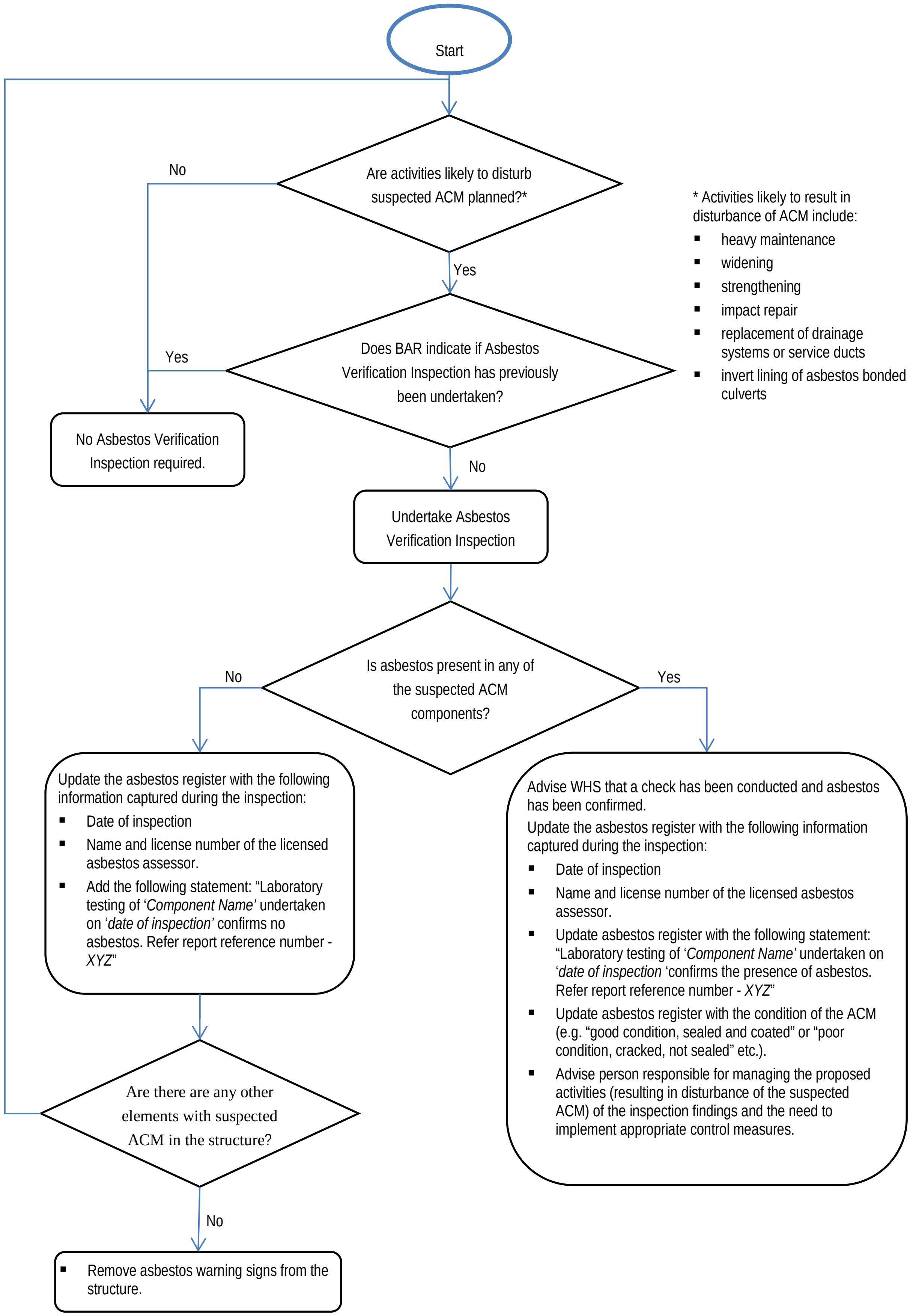
On completion of the asbestos verification inspection the following actions shall be completed:

- update the asbestos register with the following information captured during the inspection:
  - date of inspection
  - name and licence number of the licensed asbestos assessor
  - is asbestos present? (yes/no answer).
- In addition, a copy of the inspection report shall be uploaded into the BIS.
- Where no traces of asbestos are found in the suspected ACM:
  - update asbestos register with the following statement:

    Laboratory testing of [insert Element Name here] *undertaken on* [insert date of inspection here] *confirms no asbestos. Refer report reference number* [insert report reference number here].
- If there are no other elements with suspected ACM in the structure, remove asbestos warning signs from the structure.
- If asbestos is detected in the suspected ACM:
  - advise Workplace Health and Safety that a check has been conducted and asbestos has been confirmed
  - update asbestos register with the following statement:

    Laboratory testing of [insert Element Name here] *undertaken on* [insert date of inspection here] *confirms the presence of asbestos. Refer report reference number* [insert report reference number here].
  - update asbestos register with the condition of the ACM (for example, ‘good condition, sealed and coated’ or ‘poor condition, cracked, not sealed’ and so on)
  - advise person responsible for managing the proposed activities (resulting in disturbance of the suspected ACM) of the inspection findings and the need to implement appropriate control measures.

Figure 4.4.5 – Procedures for asbestos verification inspection



* Activities likely to result in disturbance of ACM include:

- heavy maintenance
- widening
- strengthening
- impact repair
- replacement of drainage systems or service ducts
- invert lining of asbestos bonded culverts

4.5 Underwater inspection

4.5.1 Background

Underwater inspections are a fundamental component of the inspection program and, as such, must be conducted for all structures located in permanent standing water at the frequency stated in Table 3.4.2 of this Manual. They should also be conducted in conjunction with the Level 2 inspection undertaken prior to the end of the defect liability period for new structures, so as to identify construction-related damage and to provide a benchmark for future inspections.

Underwater inspection of permanently submerged components are required where the submerged components cannot be satisfactorily inspected visually or by tactile means. Any structures with components meeting this criteria shall have the underwater inspection field checked in the BIS.

4.5.2 Scope

The scope of an underwater inspection is generally identical to those of an above ground Level 2 inspection as detailed in Section 3; however, there are specific requirements associated with underwater inspections which require a structure-specific plan to be developed in order to ensure that the appropriate level of detail is captured.

Underwater inspections will generally include:

- engagement of qualified divers
- mapping of local scour around piers, abutments and banks using an approved reference grid; for structures in deep water, the banks shall also be inspected 20 m upstream and downstream of the structure
- metal corrosion
- reinforced concrete cracking and spalling
- prestressed concrete splitting
- timber infestation and rot
- pile loss and residual section
- departures from designed line and level in structural members
- debris mapping
- clearance of organic growth and minor debris to permit visual inspection
- extensive photographic record achieved using local fresh water infusion or target isolation using clear polythene bags filled with fresh water
- extraction of core and other samples underwater as directed.

Additionally, tidal and splash zone areas should also be inspected and scour soundings undertaken by an accredited Level 2 inspector while access is available.

The inspection will typically be undertaken as part of a programmed Level 2 inspection, subject to the inspector being made aware of this requirement prior to the inspection in order to ensure all required resources are available.

4.5.3 Procedure

The general procedure for an underwater inspection is as follows:

- BCMAM is to draft a structure-specific brief to define the scope of works for the engagement of a diving contractor. The brief shall include:
  - a detailed scope of works outlining:
    - what is to be inspected
    - how it is to be inspected
    - supplementary testing requirements (coring and so on)
    - level of reporting required
    - inventory of standard components using Appendix C: Standard Component Identification Guidelines on **Condition Inspection Report (A2/1)** form based on the bridge drawings and other records
    - relevant standard condition state descriptions guidelines from Appendix D
    - inspection pro forma sketches
- An approved diving contractor shall be engaged to undertake works, ensuring that access equipment is sufficient for an accredited Level 2 inspector to undertake scour sounding and inspection of tidal and splash zones.
- The Contractor is to submit proposal/quotation, together with method statements, equipment schedule, key personnel schedule, work history and references (for contractors other than approved diving contractors).
- The Contractor is to complete the inspection with an accredited Level 2 inspector in attendance. The Contractor must advise the inspector of any serious defects or structural anomalies on detection and seek advice as to the need for scope changes such as coring or other sampling. It is envisaged that the inspector will be conducting the scour sounding and inspection of the tidal and splash zones while the diver undertakes the inspection.
- The accredited Level 2 inspector shall review the data collected by the diver and ensure that the required level of detail has been recorded for reporting purposes before the diver leaves site.
- The District is to contact BCMAM or other Structures Directorate section for guidance in the event that serious defects or structural anomalies are identified.
- The Contractor is to compile a draft report that includes the compilation of standard inspection forms and supplemented with a written report as required by the brief.
- The District and BCMAM are to review the report and order amendments or supplements as required.
- The District is to enter inspection reports in the BIS, comprising the condition of underwater, tidal and splash zone components, scour sounding and any material testing results.
- The District is to compile a performance report on the performance of the Contractor and forward to BCMAM.

A list of approved diving contractors is available from BCMAM on request.

4.6 Fracture critical/lack of redundancy

4.6.1 Background

There have been numerous international examples of the catastrophic collapse of bridges due to failure of members where no load redundancy exists. In response to high-profile failures in the United States, the Federal Highway Administration (FHWA) requires a special inspection of fracture critical members in fracture critical bridges. The FHWA *National Bridge Inspection Standards* (NBIS) define a fracture critical member as ‘a steel member in tension, or with a tension element, whose failure would probably cause a portion of or the entire bridge to collapse’. Essentially, the intention is to identify and inspect, to a higher level, fracture critical members in bridges with no load path redundancy.

Example structures of this type include:

- truss bridges (through, half-through, deck truss and so on)
- through girder
- tied arch
- two girder bridges
- suspension / cable-stayed spans
- drop-in/suspended spans.

4.6.2 Scope

Clearly the increased significance of the failure of fracture critical members in bridges with reduced load path redundancy justifies higher level inspection. The fracture critical/lack of redundancy inspection is intended to enhance the Level 2 inspection process detailed in Section 3.

For these structures, it is important that critical members, and likely failure indicators, are identified and mapped out on a bridge-by-bridge basis to ensure that the critical components receive the appropriate level of inspection. This may include non-destructive testing such as magnetic particle inspection or eddy current techniques as required.

A detailed inspection plan outlining inspection and testing requirements and frequency will need to be developed by BCMAM on a case-by-case basis, depending on the nature of the structure.

The inspection will typically be undertaken as part of programmed Level 2 inspection, subject to the inspector being made aware of this requirement prior to the inspection, in order to ensure all required resources are available.

4.6.3 Procedures and inspector accreditation

Fracture critical/lack of redundancy inspections shall be carried out by or under the supervision of an experienced RPEQ bridge engineer.

All specialist inspection activities must be undertaken by suitably qualified and experienced practitioners. All testing must be undertaken by parties with appropriate NATA accreditation (where applicable).

The inspection procedure will be as specified by the structure-specific inspection plan contained within the SMP1.

4.6.4 Inspection frequency

The inspection frequency will be as specified in the structure-specific inspection plan contained within the SMP.

4.6.5 Extent of inspections

The extent of the inspection will be in accordance with the structure-specific inspection plan.

4.6.6 Data recording in the field

Data recording requirements will be in accordance with the structure-specific inspection plan.

4.6.7 Reporting

Reporting will be in accordance with the structure-specific inspection plan.

4.7 Sub-standard load rating

4.7.1 Background

Transport and Main Roads has recently completed load ratings on all bridges on their 'B-double and road train' routes. This involved high-level assessment of all bridges with grillage analysis undertaken on a representative sample. This rating exercise identified a potentially large number of structures with a theoretical sub-standard load rating and also highlighted potential detailing issues and / or lack of redundancy in structural form.

4.7.2 Scope

A detailed scope for the Level 3 investigation and inspection of structures with a theoretical sub-standard load rating is presented in *Level 3 Inspection Criteria for Potentially Structurally Deficient Bridges* (Issue 1.06, May 2014).

4.7.3 Procedures and inspector accreditation

Outline procedures and inspector accreditation are outlined in *Level 3 Inspection Criteria for Potentially Structurally Deficient Bridges* (Issue 1.06, May 2014).

4.8 Complex/unique structures

4.8.1 Background

Most unique, complex structures require more detailed reporting than the standard Level 2 inspection report allows. Guidance on enhancing Level 2 inspections for complex structures is provided in Appendix G of this Manual; however, the specific inspection requirements / procedures and additional inspector skills / experience required for such structures are not covered indepth.

Complex structures are usually those that, because of their size or complexity, require significantly greater inspection effort / resource than a normal Level 2 inspection. Typically, specialised access equipment and / or greater engineering knowledge will be required to accurately determine the condition of various components. Examples of complex structures include:

- suspension
- cable stayed
- curved box girders

- large bridges comprising multiple structure forms (for example, Riverside Expressway) or complex components (for example, mechanical pot bearings, pin and hanger connections, cable systems and so on) with specific servicing / inspection requirements.

4.8.2 Scope

Any structure classified as ‘complex’ should have its own detailed inspection plans (developed by BCMAM), similar to fracture critical/low redundancy structures, which define areas requiring enhanced access/inspection requirements and appropriate inspection frequencies.

The inspection will typically be undertaken as part of a programmed Level 2 inspection, subject to the inspector being made aware of this requirement prior to the inspection, in order to ensure all required resources are available.

Most complex and unique structures are outside the maintenance funding of Element 19; hence the District is required to arrange special funding from other sources for routine, scheduled and special maintenance of these structures.

Some bridges of these bridges require repainting. The cost of repainting one of these bridges is large, requires special funding and needs to be planned some years prior to repainting.

4.8.3 Procedures and inspector accreditation

Complex/unique structure inspections shall be carried out by or under the supervision of an experienced RPEQ bridge engineer.

All specialist inspection activities must be undertaken by suitably qualified and experienced practitioners. All testing must be undertaken by parties with appropriate NATA accreditation (where applicable).

The inspection procedure will be as specified by the structure-specific inspection plan contained within the SMP.

4.8.4 Inspection frequency

The inspection frequency will be as specified in the structure-specific inspection plan contained within the SMP.

The specialist inspection and the associated inspection frequency needs to be documented in the SMP.

Complex and unique bridges often have their own gantries and equipment. The maintenance of this equipment is critical from both a safety and operational perspective and needs to be documented.

4.8.5 Extent of inspections

The extent of the inspection will be in accordance with the structure-specific inspection plan.

4.8.6 Data recording in the field

Data recording requirements will be in accordance with the structure-specific inspection plan.

4.8.7 Reporting

Reporting will be in accordance with the structure-specific inspection plan.

4.8.8 Budget

Most complex and unique bridges do not receive maintenance funding from Element 19. Consequently, it is necessary to prepare a budget with a duration of:

- 50 years if the bridge has bearings
- 40 years if the bridge requires painting
- 20 years if there is no painting and no bearings.

4.8.9 Example for requirements for complex/unique bridges

An example of the requirement for complex and unique bridges is shown in Appendix H.

4.9 Known/suspected deficiencies

4.9.1 Background

Another of the outcomes of the ‘B-double and road train’ exercise was the identification of structure types / families with possible deficiencies (design or detailing) or a lack of redundancy. These structure types include:

- portal frame piers/abutments
- headstocks with cantilevers > 0.7 m in length
- pre-1965 precast deck units with span = 8.23 m (27’)
- simply supported spans > 30 m and designed to MS18 + 25% or T44 design loading
- shear and torsion related issues in all concrete box girder bridges
- bridges with shear reinforcement significantly less than the current code minimum.

4.9.2 Scope

A detailed scope for the Level 3 investigation and inspection of structures with a theoretical sub-standard load rating is presented in *Level 3 Inspection Criteria for Potentially Structurally Deficient Bridges* (Issue 1.06, May 2014).

4.9.3 Procedures and inspector accreditation

Outline procedures and inspector accreditation are outlined in *Level 3 Inspection Criteria for Potentially Structurally Deficient Bridges* (Issue 1.06, May 2014).

4.10 Confined space inspection

4.10.1 Background

Confined spaces pose dangers because they are usually not designed to be areas where people work. Confined spaces often have poor ventilation which allows hazardous atmospheres to quickly develop, especially if the space is small. The hazards are not always obvious and may change from one entry into the confined space to the next.

The risks of working in confined spaces include:

- loss of consciousness, impairment, injury or death due to the immediate effects of airborne contaminants
- fire or explosion from the ignition of flammable contaminants

- difficulty rescuing and treating an injured or unconscious person
- asphyxiation resulting from oxygen deficiency or immersion in a free-flowing material, such as liquids, grain, sand, fertiliser or water.

A confined space means an enclosed or partially enclosed space that:

- is not designed or intended primarily to be occupied by a person
- is, or is designed or intended to be, at normal atmospheric pressure while any person is in the space and
- is or is likely to be a risk to health and safety from:
  - an atmosphere that does not have a safe oxygen level
  - contaminants, including airborne gases, vapours and dusts, that may cause injury from fire or explosion
  - harmful concentrations of any airborne contaminants, or
  - engulfment.

4.10.2 Scope

The scope of confined space inspection is generally identical to those of an above ground Level 2 inspection as detailed in Section 3; however, as the space has been determined to be a ‘Confined Space’ under the Workplace Health and Safety Regulation 2011, there are specific requirements associated with these inspections which require a structure-specific plan to be developed in order to ensure that the appropriate risk assessment, management and control procedures are implemented.