### APPENDIX A - DATA SHEETS FOR LEVEL 2 INSPECTION

1. **General**

2. **Data sheets for bridges and culverts**

3. **Data sheets for roadside structures**

### APPENDIX B – DETERIORATION OF ROAD STRUCTURES

<table>
<thead>
<tr>
<th>1. <strong>Material defects</strong></th>
<th>368</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1 General</td>
<td>368</td>
</tr>
<tr>
<td>1.2 Concrete</td>
<td>368</td>
</tr>
<tr>
<td>1.2.1 Scaling</td>
<td>368</td>
</tr>
<tr>
<td>1.2.2 Disintegration</td>
<td>369</td>
</tr>
<tr>
<td>1.2.3 Water wash</td>
<td>369</td>
</tr>
<tr>
<td>1.2.4 Corrosion of reinforcement</td>
<td>370</td>
</tr>
<tr>
<td>1.2.5 Delamination</td>
<td>371</td>
</tr>
<tr>
<td>1.2.6 Spalling</td>
<td>371</td>
</tr>
<tr>
<td>1.2.7 Cracking</td>
<td>372</td>
</tr>
<tr>
<td>1.2.8 Alkali aggregate reaction (AAR)</td>
<td>374</td>
</tr>
<tr>
<td>1.2.9 Surface defects</td>
<td>374</td>
</tr>
<tr>
<td>1.2.10 Carbonation</td>
<td>375</td>
</tr>
<tr>
<td>1.2.11 Chloride ingress</td>
<td>375</td>
</tr>
<tr>
<td>1.3 Steel</td>
<td>375</td>
</tr>
<tr>
<td>1.3.1 Corrosion</td>
<td>375</td>
</tr>
<tr>
<td>1.3.2 Permanent deformations</td>
<td>376</td>
</tr>
<tr>
<td>1.3.3 Cracking</td>
<td>377</td>
</tr>
<tr>
<td>1.3.4 Loose connections</td>
<td>379</td>
</tr>
<tr>
<td>1.4 Timber</td>
<td>379</td>
</tr>
<tr>
<td>1.4.1 Fungal Rot</td>
<td>380</td>
</tr>
<tr>
<td>1.4.2 Termites</td>
<td>380</td>
</tr>
<tr>
<td>1.4.3 Marine organisms</td>
<td>381</td>
</tr>
<tr>
<td>1.4.4 Corrosion of fasteners</td>
<td>382</td>
</tr>
<tr>
<td>1.4.5 Shrinkage and splitting</td>
<td>382</td>
</tr>
<tr>
<td>1.4.6 Fire Damage</td>
<td>383</td>
</tr>
<tr>
<td>1.4.7 Flood Damage</td>
<td>383</td>
</tr>
<tr>
<td>1.4.8 Weathering</td>
<td>383</td>
</tr>
<tr>
<td>1.5 Masonry</td>
<td>383</td>
</tr>
<tr>
<td>1.5.1 Cracking</td>
<td>384</td>
</tr>
<tr>
<td>1.5.2 Splitting, spalling and disintegration</td>
<td>384</td>
</tr>
<tr>
<td>1.5.3 Loss of mortar and stones</td>
<td>384</td>
</tr>
<tr>
<td>1.5.4 Arch stones dropping</td>
<td>385</td>
</tr>
<tr>
<td>1.5.5 Side wall movement at masonry arch</td>
<td>385</td>
</tr>
<tr>
<td>1.5.6 Deformation</td>
<td>385</td>
</tr>
<tr>
<td>1.5.7 Separation of arch rings</td>
<td>385</td>
</tr>
<tr>
<td>1.6 Protective coatings</td>
<td>386</td>
</tr>
<tr>
<td>1.7 Fibre reinforced polymer strengthening</td>
<td>386</td>
</tr>
</tbody>
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<table>
<thead>
<tr>
<th>1.2 - Common causes of bridge deterioration</th>
<th>386</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2.1 Concrete bridges</td>
<td>386</td>
</tr>
<tr>
<td>1.2.1.1 Monolithic and simply supported T-beams</td>
<td>387</td>
</tr>
<tr>
<td>1.2.1.2 Precast I beams</td>
<td>387</td>
</tr>
<tr>
<td>1.2.1.3 Precast prestressed inverted T beams</td>
<td>387</td>
</tr>
<tr>
<td>1.2.1.4 Box-girder bridges</td>
<td>387</td>
</tr>
</tbody>
</table>
1.2.1.5 Prestressed voided flat slab bridges .................................................................................................................. 388
1.2.1.6 Reinforced concrete flat slab bridges .................................................................................................................. 388
1.2.1.7 Rail in slab concrete bridges .............................................................................................................................. 388
1.2.1.8 Precast prestressed slabs ......................................................................................................................................... 388
1.2.1.9 Precast U slabs .......................................................................................................................................................... 389
1.2.1.10 Precast prestressed voided T slabs .......................................................................................................................... 400
1.2.1.11 Decks and overlays .................................................................................................................................................. 400
1.2.1.12 Diaphragms ............................................................................................................................................................. 400
1.2.1.13 Kerbs, footways, posts and railing .......................................................................................................................... 400
1.2.1.14 Abutments ............................................................................................................................................................... 401
1.2.1.15 Piers ........................................................................................................................................................................... 402
1.2.2 Steel bridges ............................................................................................................................................................... 402
1.2.3 Timber bridges ............................................................................................................................................................. 403
1.2.3.1 Timber stringers ......................................................................................................................................................... 403
1.2.3.2 Corbels and corbel blocks ......................................................................................................................................... 404
1.2.3.3 Decking ....................................................................................................................................................................... 404
1.2.3.4 Kerbs, posts and railing .............................................................................................................................................. 405
1.2.3.5 Piles ................................................................................................................................................................................ 406
1.2.3.6 Walings and crossbraces ............................................................................................................................................. 406
1.2.3.7 Crossheads ................................................................................................................................................................. 406
1.2.3.8 Abutments ................................................................................................................................................................. 407
1.2.4 Deck joints ....................................................................................................................................................................... 407
1.2.4.1 General ............................................................................................................................................................................ 407
1.2.4.2 Inspection of Deck Joints ......................................................................................................................................... 408
1.2.5 Step (Half) Joints ........................................................................................................................................................... 409
1.2.6 Bearings ........................................................................................................................................................................... 410
1.2.7 Culverts ............................................................................................................................................................................... 411
1.2.7.1 Concrete box culverts .................................................................................................................................................. 411
1.2.7.2 Concrete pipe culverts .................................................................................................................................................. 411
1.2.7.3 Masonry arch culverts .................................................................................................................................................. 411
1.2.7.4 Buried Corrugated Metal Structures (BCMS) - pipes and arch culverts ...................................................................... 411
1.2.8 Causes of deterioration not related to bridge materials ................................................................................................. 414
1.2.8.1 Damage due to accidents ............................................................................................................................................ 414
1.2.8.2 Drainage ........................................................................................................................................................................ 415
1.2.8.3 Debris ................................................................................................................................................................................ 415
1.2.8.4 Vegetation ....................................................................................................................................................................... 415
1.2.8.5 Scouring of foundations ............................................................................................................................................... 415
1.2.8.6 Movement of the structure .......................................................................................................................................... 416
1.2.8.7 Condition of approach embankments .......................................................................................................................... 416
1.2.9 Deterioration of roadside structures ............................................................................................................................. 416
1.2.9.1 Major sign structures ................................................................................................................................................... 417
1.2.9.2 High mast lighting ......................................................................................................................................................... 417
1.2.9.3 Noise attenuation and visual screen walls .................................................................................................................... 417
1.2.9.4 Retaining walls ............................................................................................................................................................. 418
1.2.9.5 Emergency median barrier access gates .................................................................................................................... 418
Appendix A - Data sheets for level 2 inspection

1.1 General

It is essential that uniformity be developed for the inspection of the VicRoads assets so results from different inspectors are based on the same set of standards. The field sheets shown in this section of the manual aim to achieve such a uniform approach.

1.2 Data sheets for bridges and culverts

- Inventory and photographic record sheet for SN - bridge or culvert
- Bridge inspector’s sheet
- Condition rating sheet
- Structure defect sheet
- Structure information sheet
- Sketch sheet

1.3 Data sheets for roadside structures

- The relevant inventory and photographic record sheet shall be used for the following roadside structures.
- SS - Major sign structures
- SL - High mast lighting structures
- SZ - Noise attenuation walls
- SR - Retaining walls
- SV - Visual screen walls

Then the same sheets as for bridges and culverts shall be used for recording component condition and other general information:

- Bridge inspector’s sheet
- Condition rating sheet – general
- Structure defect sheet
- Structure information sheet
- Sketch sheet
# VicRoads Level 2 structure condition inspection

## Bridge inspector’s sheet

<table>
<thead>
<tr>
<th>Structure ID No.: S</th>
<th>Location (Km): (Fwd/Rev)</th>
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<td>Road name:</td>
<td>Road number:</td>
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<td>Crossing/General Location:</td>
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<tr>
<td>Region:</td>
<td>Map reference: (Melways/VicRoads)</td>
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<td>Inspector:</td>
<td>Date:</td>
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## Site conditions

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<th>Climatic</th>
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## Equipment requirements for next inspection

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<thead>
<tr>
<th>Inspection Access</th>
<th>Tools</th>
<th></th>
<th>Tools</th>
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<tbody>
<tr>
<td>Under-bridge inspection Unit</td>
<td>Hammer</td>
<td>Torch</td>
<td></td>
<td></td>
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<tr>
<td>Scissor Lift</td>
<td>Probe</td>
<td>Binoculars</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Boom lift</td>
<td>GPS unit</td>
<td>Measuring tapes</td>
<td></td>
<td></td>
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<tr>
<td>Ladder</td>
<td>Electronic Camera</td>
<td>Crack Gauge</td>
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<tr>
<td>Boat</td>
<td>Measuring Wheel</td>
<td>Traffic control</td>
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<tr>
<td>Other _</td>
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Note: Personal safety and protective equipment for use on roadway and in 1m water is mandatory.

## General comments

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Sheet of
## VicRoads Level 2 structure condition inspection

### Condition rating sheet

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<tr>
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<th>Location (Km): (Fwd/Rev)</th>
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<td>Map reference: (Melways/VicRoads)</td>
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<td>Inspector:</td>
<td>Date:</td>
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<table>
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<tr>
<th>Component</th>
<th>Widening</th>
<th>% of component in each condition</th>
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<td>No.</td>
<td>L/R</td>
<td>1/2</td>
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<td>1</td>
<td></td>
<td>2</td>
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<tr>
<td>2</td>
<td></td>
<td>3</td>
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<td>3</td>
<td></td>
<td>4</td>
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</table>

### Notes

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| Sheet of |
VicRoads Level 2 structure condition inspection

Structure defect sheet

<table>
<thead>
<tr>
<th>Component No.</th>
<th>Component Name</th>
<th>Location</th>
<th>Quantity</th>
<th>Photo Nos.</th>
<th>Defect description</th>
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Structure ID No.: S  
Location (Km): (Fwd/Rev)

Road name:  
Road number:

Crossing/General Location:

Region:  
Map reference: (Melways/VicRoads)

Inspector:  
Date:

Sheet of
## VicRoads Level 2 structure condition inspection

### Structure information sheet

<table>
<thead>
<tr>
<th>Component No.</th>
<th>Information or comment (Including: Load, Height, speed limits; hydraulic performance or similar; and location of any material testing &amp; sampling)</th>
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**Structure ID No.:** S  
**Location (Km):** (Fwd/Rev)

**Road name:**  
**Road number:**

**Crossing/General Location:**

**Region:**  
**Map reference:** (Melways/VicRoads)

**Inspector:**  
**Date:**

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| Sheet of |  |
VicRoads Level 2 structure condition inspection

**Sketch sheet**

<table>
<thead>
<tr>
<th>Structure ID No.: S</th>
<th>Location (Km): (Fwd/Rev)</th>
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<td>Map reference: (Melways/VicRoads)</td>
</tr>
<tr>
<td>Inspector:</td>
<td>Date:</td>
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</table>
VicRoads Level 2 structure condition inspection

**Inventory and photographic record sheet**  
- **Bridge (SN)**  
- **Culvert (SN)**

<table>
<thead>
<tr>
<th>Structure ID No.</th>
<th>Location (Km): (Fwd/Rev)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Road name:</td>
<td>Road number:</td>
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<td>Map reference: (Melways/VicRoads)</td>
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<tr>
<td>Region:</td>
<td>Inspector:</td>
</tr>
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<td></td>
<td>Date:</td>
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</table>

**GPS**

<table>
<thead>
<tr>
<th>Latitude South:</th>
<th>Longitude East:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location:</td>
<td></td>
</tr>
</tbody>
</table>

**Bridge or Culvert measurements and quantities (1 = 1<sup>st</sup> widening on one side; 2 = 2<sup>nd</sup> widening on one side)**

<table>
<thead>
<tr>
<th>Original structure</th>
<th>B: Bridge, C: Culvert (select one &amp; cross the other)</th>
<th>Widening left</th>
<th>Widening right</th>
<th>Whole structure</th>
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<tbody>
<tr>
<td></td>
<td>B: Length (m)</td>
<td>1 2</td>
<td>1 2</td>
<td>Width between kerbs: (m)</td>
</tr>
<tr>
<td></td>
<td>C: Cell length/dia (m)</td>
<td></td>
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<tr>
<td></td>
<td>B: O/All width (m)</td>
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<td></td>
<td>C: Cell width along invert (m)</td>
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<tr>
<td></td>
<td>B: No. spans</td>
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<td>Width between kerbs: (m)</td>
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<td>C: Cell height</td>
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<tr>
<td></td>
<td>B: No. beams/slabs</td>
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<tr>
<td></td>
<td>C: No. of cells</td>
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<th>Span No.</th>
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<td>B: Span Length (m)</td>
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<td>C: Cell Size (m)</td>
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**Notes**

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**Sheet of**
VicRoads Level 2 structure condition inspection

Inventory and photographic record sheet

[ ] Major sign structure (SS)  [ ] High mast lighting (SL)

<table>
<thead>
<tr>
<th>Structure ID No.:</th>
<th>Location (Km): (Fwd/Rev)</th>
</tr>
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<tr>
<td>Road name:</td>
<td>Road number:</td>
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Crossing/General Location:

<table>
<thead>
<tr>
<th>Region:</th>
<th>Map reference: (Melways/VicRoads)</th>
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Inspector: Date:

GPS

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<th>Latitude</th>
<th>Longitude</th>
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<tr>
<td>South:</td>
<td>East:</td>
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Location:

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<thead>
<tr>
<th>Side of road: (left or right)</th>
<th>Clearance from carriageway: m</th>
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ONLY FOR MAJOR SIGN STRUCTURE

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<thead>
<tr>
<th>Type of Sign:</th>
<th>Cantilever</th>
<th>Butterfly</th>
<th>Gantry</th>
<th>Pedestal</th>
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<tr>
<td>VMS:</td>
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<td>No</td>
<td>Other</td>
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<table>
<thead>
<tr>
<th>Base</th>
<th>Located on concrete pad</th>
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<th>No</th>
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<tbody>
<tr>
<td>Levelling nuts exist</td>
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Photos

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General comments

Sheet of
### VicRoads Level 2 structure condition inspection

**Inventory and photographic record sheet**
- Visual Screen Walls (SV)
- Noise Attenuation Walls (SZ)
- Retaining Walls (SR)

<table>
<thead>
<tr>
<th>Structure ID No.</th>
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<td>Crossing/General Location:</td>
<td>Municipality:</td>
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<td>Region:</td>
<td>Melways/VicRoads Map Ref:</td>
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<td>Inspector:</td>
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**GPS start of wall (lowest chainage)**

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<th>Latitude South:</th>
<th>Longitude East:</th>
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<tr>
<td>Overall length (if requested):</td>
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**For Visual Screen or Noise Attenuation Wall**

- Type of wall: (freestanding, on parapet or retaining wall, other)
- Material: (steel, concrete, timber, masonry, other)

**For Retaining Wall**

- Materials: Facing, Supports

<table>
<thead>
<tr>
<th>Side of road: (left or right)</th>
<th>Clearance from carriageway: m</th>
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**Photos**

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**General comments**

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**For retaining walls a sketch shall be provided showing each segment with start chainage, length, end chainage using Sketch Sheet above.**
Appendix B – Deterioration of road structures

1.1 Material defects

1.1.1 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.

1.1.2 Concrete
Concrete elements may be unreinforced mass concrete, reinforced concrete or prestressed concrete. This section is based on concrete defects described in Ontario Ministry of Transportation, Ontario Structure Inspection Manual.

Defects in concrete are commonly linked with poor durability resulting from the composition of the concrete, poor workmanship and quality control during construction and/or the aggressive environment surrounding and in contact with the structure.

The following defects commonly occur in concrete:

- Scaling
- Disintegration
- Water wash
- Corrosion of reinforcement
- Delamination
- Spalling
- Cracking
- Alkali Aggregate Reaction
- Surface Defects
- Carbonation
- Chloride ingress

1.1.2.1 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 1.1.2.1.
1.1.2.2 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 1.1.2.2.

1.1.2.3 Water wash

Water wash is caused by water borne sand and gravel particles eroding the concrete surface. Water wash of a concrete column is shown in Figure 1.1.2.3.
1.1.2.4 Corrosion of reinforcement

Corrosion is the deterioration of reinforcement by the process of oxidation. Corrosion can also occur in the presence of high Chloride ion concentration such as when the concrete is immersed in sea-water or exposed to salt-spray. Corrosion may appear as a rust stain on the concrete surface initially. In the advanced stages, the surface concrete above the reinforcement can crack, delaminate and spall-off exposing the underlying reinforcement. This process is illustrated in Figure 1.1.2.4(a) & (b)
1.1.2.4(b) Corrosion of reinforcement in concrete

1.1.2.5 Delamination

Delamination is defined as a discontinuity in the surface concrete which is substantially separated but not completely detached from the main mass of concrete. Visibly, the concrete may appear to have a solid surface, however, the delamination can be identified by the hollow sound if the concrete is tapped with a hammer. Delamination commonly begins with the corrosion of reinforcement and subsequent cracking of the concrete and normally occurs in the plane of the reinforcement parallel to the exterior surface of the concrete. It can also result from impact and from crushing that occurs when two concrete components come into contact.

1.1.2.6 Spalling

A spall is a fragment, which has been detached from a larger concrete mass. Spalling is a continuation of the delamination process in which the pressure exerted by the corrosion of reinforcement results in complete separation of the delaminated concrete.

Vehicular or other impact can also result in spalling. Spalling may also be caused by overloading of the concrete in compression. Spalling may also occur in areas of localised high compressive load concentrations, such as at structure supports, or at anchorage zones in prestressed concrete. Concrete exposed to extreme temperatures such as in a fire may also spall.

The spalled area left behind is characterised by irregular edges.

Spalling of concrete is shown in Figure 1.1.2.6(a) & (b).
1.1.2.7 Cracking

A crack is a fracture in the concrete which extends partly or completely through the member. Cracks in concrete are caused by tensile stresses induced in the concrete as result of volumetric changes or loads applied. Concrete is weak in tension. When the level of the tensile stress in concrete exceeds its tensile capacity, the concrete cracks. After this point the tensile force is transferred to the steel reinforcement. The purpose of reinforcement and prestressed strand is to control crack width and crack distribution.

Tensile stresses and cracks in concrete may be due to externally applied loads, external restraint forces, internal restraint forces, differential movement and settlements, or corrosion of the reinforcement. Externally applied loads generate compressive and tensile stresses in the members and components of the structure. Cracks resulting from externally applied loads initially appear as hairline cracks and are not significant. However, if the applied load increases, the stress in the reinforcement rises and the initial cracks widen and progressively spread.

Cracks may also be caused by:

- external restraint forces if the free movement of the concrete arising from temperature, creep and shrinkage is prevented
- internal restraint forces resulting from the differential expansion or contraction of the exterior surface of concrete relative to the interior mass of the concrete - e.g. plastic shrinkage cracking and early thermal cracking. The resulting surface cracks are normally shallow and appear as pattern cracks
- differential movements or settlements resulting 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 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.

The types and location of cracking that are the most likely to be observed are shown in Figure 1.1.2.7 (a).
Figure 1.1.2.7 (a) Types and location of cracks in concrete structures

The severity of cracking is shown in Figure 1.1.2.7 (b) and is defined as:

- **Hairline** - up to 0.1 mm
- **Fine** - 0.1 to 0.3 mm
- **Medium** - 0.3 to 0.7 mm
- **Heavy** - > 0.7 mm.

![Crack Types and Location Diagram](image)

Figure 1.1.2.7 (b)

When the concrete surface around a crack has spalled, it is important to ensure that the actual crack width is measured rather than the spalled width as shown in Figure 1.1.2.7 (c).

![Crack Sizes Diagram](image)
1.1.2.8 Alkali aggregate reaction (AAR)

Some aggregates react adversely with the alkalis in cement to produce a highly expansive alkali-silica gel. The expansion of the gel and aggregates under moist conditions leads to cracking and deterioration of the concrete. The cracking occurs through the entire mass of the concrete. AAR is generally slow by nature, and the results may not be apparent for many years. The appearance of concrete affected by alkali-aggregate reactions is shown in Figure 1.1.2.8.

1.1.2.9 Surface defects

The following are examples of surface defects in concrete:

- Segregation
- Cold Joints
- Surface deposits - efflorescence, stalactite
- Honeycombing
- Abrasion and wear

Surface defects are not necessarily serious. However, they can be indicative of a potential weakness in the concrete.

Segregation is the differential concentration of the components in fresh concrete resulting in variable composition. For example, when concrete is allowed to fall from a height of more than 2m, the coarse aggregate may settle to the bottom of the fresh concrete mass leaving an excess of the fine particles at the upper part of the mass.
Other causes of segregation are poor mix design or if closely spaced reinforcing bars prevent the uniform flow of concrete.

Cold Joints are produced if there is a delay between the placement of successive deliveries of concrete, and if an incomplete bond develops at the joint due to the partial setting of concrete in the first pour.

Deposits are often left behind where water percolates through the hardened concrete and dissolves or leaches chemicals from it and deposits them on the surface.

Deposits may appear as the following:
- efflorescence - a deposit of salts (chemical components of the concrete), usually white and powdery refer to Figure 1.1.2.9
- exudation - a liquid or gel-like discharge through pores or cracks in the surface
- encrustation - a hard crust or coating formed on the concrete surface
- stalactite - a downward pointing formation hanging from the concrete surface, usually shaped like an icicle and made from salts in the concrete.

Honeycombing is caused by inadequate compaction of the concrete which results in voids where the cement matrix failed to completely fill the spaces between the coarse aggregate.

Abrasion damage is caused by contact with vehicles and results in the removal of the concrete surface. It can also be caused by friction of water-borne particles against submerged members. This phenomenon is also known as water wash.

Slippery surfaces - e.g. polished concrete deck - may be caused by the repetitive passage of vehicles.

1.1.2.10 Carbonation
Carbonation is a process through which the alkalinity of concrete slowly reduces over time due to the ingress of atmospheric carbon-dioxide. Reduction in alkalinity leads to corrosion of embedded steel reinforcement.

1.1.2.11 Chloride ingress
Sodium chloride in the atmosphere or in water can penetrate concrete through to the reinforcement. The sodium chloride separates into Sodium (Na) and Chloride (Cl) ions. When the chloride ions reach the reinforcing steel, corrosion of the embedded reinforcement occurs. Corroding steel increases in volume leading to the risk of delamination and spalling. The greatest risk of Chloride ion ingress occurs in coastal areas and in river estuaries where tidal flows can bring salt-laden (brackish) water inland. Salt spray may be blown inland by strong winds affecting structures several kilometres from the sea.

1.1.3 Steel
Based on Ontario Ministry of Transportation, Ontario Structure Inspection Manual.

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

The following defects commonly occur in steel:
- Corrosion
- Permanent Deformations
- Cracking
- Loose connections

1.1.3.1 Corrosion
Corrosion (rust) is the oxidation of steel resulting from exposure to air, moisture, fumes, chemicals and contact with other metals. Corrosion can be prevented or minimised by the use of coatings but the effectiveness of these coatings is reduced or lost if the coating is damaged.

Rust on carbon steel is initially fine grained, but as rusting progresses it becomes flaky and delaminates, exposing a pitted surface leading to a progressive loss of section.
Light corrosion can be identified as small reddish brown spots and occurs on steelwork when the existing protective coating is loss.

![Figure 1.1.3.1 Light corrosion on steel work and loss of protective coating](image1)

Pitting corrosion can be identified as holes and cavities and is caused by localised corrosion. This generally progresses from light corrosion spots on steelwork or is due to localised water ponding or exposure.

Section loss can be identified as uniform area of steelwork with noticeable cross sectional loss. This may occur when the section of steel work has lost its protective coating exposing the steel surface area to environmental conditions.

![Figure 1.1.3.2 Section Loss and Pitting Through Steel](image2)

### 1.1.3.2 Permanent deformations

Permanent deformation of steel members can take the form of bending, buckling, twisting or elongation, or any combination of these. Permanent deformations may be caused by overloading, vehicular collision, foundation settlement or inadequate or damaged intermediate lateral supports or bracing.

Permanent bending deformation generally occurs in flexural members in the direction of the applied loads. However, vehicular impact may produce permanent bending deformation in any member.
Permanent buckling deformation generally occurs in compression members in a direction perpendicular to the applied load. Buckling may also produce local permanent deformations of webs and flanges of beams, plate girders and box girders.

Permanent twisting is a rotation of the member about its longitudinal axis and usually results from eccentric transverse loads on the member.

Permanent axial deformation occurs along the length of the member and is normally associated with tensile loads.

1.1.3.3 Cracking

Cracking is a linear fracture of the steel and is normally caused by fatigue. It can lead to brittle fracture of the affected component and to more widespread structural failure.

Brittle fracture is a crack completely through the component that usually occurs without plastic deformation and with little or no warning. Brittle fracture may result at fatigue prone details after initial fatigue cracking.

The primary factors leading to fatigue cracking are:

- The number of applied stress cycles (influenced by volume of traffic and/or the wind loading and the effects of passing vehicles)
- The magnitude of the stress range which depends on the applied live load
- The fatigue resistance of the connection detail (which is influenced by the strength, toughness and geometry of the components and the weld size and geometry.

Fatigue cracks normally occur at points of tensile stress concentrations, at welded attachments or at termination points of welds in components subject to cyclic loading. Cracks may also be caused or enlarged by overloading, vehicular collision or loss of section thickness due to corrosion. Poorly designed and fabricated details and the fracture toughness of the steel are also contributing factors. Fracture toughness determines the size of the crack that can be tolerated before fracture occurs.

Welded components are more susceptible to cracking than bolted or riveted components. If cracking occurs in a welded connection, it can extend into other components and possibly lead to a brittle fracture.

Bolted or riveted connections may also develop fatigue cracks, 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 as a result of the force generated by expansive corrosion between connection components.

Common locations susceptible to cracking are illustrated in Figure 1.1.3.3(a) & (b). As cracks may be concealed by rust, dirt or debris, the surfaces should be cleaned prior to inspection.

Cracks that are perpendicular to the direction of stress are potentially very serious; those parallel to the direction of stress less so. In either case, cracks in steel components should be treated with caution as parallel cracks may for a number of reasons turn into a perpendicular crack. Any crack should be carefully noted and recorded including its specific location in the member, and the member’s location in the structure. The length, width and direction of crack should also be recorded.
Figure 1.1.3.3(a) Common locations susceptible to cracking
1.1.3.4 Loose connections

Bolted and riveted connections may become loose as a result of corrosion of the connector plates or fasteners, excessive vibration, overstressing, cracking, or the failure of individual fasteners. Loose connections may sometimes be undetectable 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. If a connection is suspect but the status cannot be confirmed, its full details should be noted as required for cracks in welds and further monitoring/investigation should be arranged.

1.1.4 Timber

Timber bridges were extensively used on Victorian roads until the middle 1900s and now constitute only a small proportion of the structures on the state road network. The majority of timber bridges are on local roads controlled by municipalities. Some are on tourist roads and forest roads and may carry heavy loads.

The following defects commonly occur in timber bridge components:

- Fungal rot
- Termites
- Marine organisms
- Corrosion of fasteners
• Shrinkage and splitting
• Fire damage
• Flood damage
• Weathering

This section is based on Austroads 1991 Bridge Management Practice.

1.1.4.1 Fungal Rot

White rot or brown rot fungi causes severe internal decay of bridge timbers members. External surface decay, especially in ground contact areas, is caused by soft rot fungi. Other fungi such as mould and sap stain fungi may produce superficial discolouration on timbers but are not generally of structural significance.

Fungal growth does not occur unless there is a source of infection from which the fungus can grow. Fungi procreate by producing vast numbers of microscopic spores which will not germinate and develop unless there is:

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

Once established and provided that favourable conditions prevail, the decay fungi continue to grow at an accelerating rate. Depriving the fungi of any one of the 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 1.1.4.1 Pile failure resulting from heartwood rot.

1.1.4.2 Termites

Australia has a large number of termite species which are widely distributed. Heavy termite attack is found in the northern tropical belt of Australia but the hazard is sufficient in southern Australia to constitute a significant problem. 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.

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 reproductive
termites 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. 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. 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 200m from the original nest. After migration, their wings fall off and a few may pair to start new colonies.

Well-established termite attack usually degrades timber much more quickly than fungi, but it is rare for termite attack to occur in durable hardwoods normally used in bridge construction without 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 most of the material removed by termites has already lost its structural strength because of decay, the control of termites remains an 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.

Refer to Figure 1.1.4.2 showing termite attack.

![Figure 1.1.4.2 Termite attack](image)

### 1.1.4.3 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 (teredinidae) - this group includes various species of Teredo, Nausitora and Bankia.
- Crustaceans - this group includes species of Sphaeroma (pill bugs), Limnoria (gribbles), and Chelura.

Teredinid molluscs 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 siphons remains at the original entrance. The teredine borers destroy timber at all levels from the midline to high water level, but the greatest intensity of the attack occurs in the zone between 300mm above and 600mm 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 to Figure 1.1.4.3 for signs of Teredinid marine borer.
Figure 1.1.4.3 Signs of teredinid marine borer.

Crustaceans attack the wood on its surface, making many narrower and shorter tunnels than those made by the teredinines. The timber so affected is steadily eroded from the outside by wave action and the piles assume a wasted appearance or hourglass effect. Attack by Sphaeroma 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 per cent salinity, but can grow at lower temperatures than the teredinines.

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.

1.1.4.4 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).

1.1.4.5 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.

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.

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 connection is therefore a standard item of routine maintenance for timber bridges.
1.1.4.6 Fire Damage

References include Bootle (1983)

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 20°C (at the rate of 1% per century). At 93°C the wood will become charred in about 5 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: 95°C - 200°C water vapour is given off and wood eventually becomes charred
- zone B: 200°C - 280°C water vapour, formic and acetic acids and glyoxal are given off, ignition is possible but difficult
- zone C: 280°C - 500°C 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 500°C, 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: 500°C - 1000°C in this zone the char develops the crystalline structure of graphite, glowing occurs and the char is gradually consumed
- zone E: above 1000°C 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 280°C.

1.1.4.7 Flood Damage

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.

A prime example of flood damage was the 1946 floods in the Western District when approximately 13 major timber structures were washed away.

A special inspection of all structures is required following a major flood event.

1.1.4.8 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 and end splitting occurs.

1.1.5 Masonry

Based on Ontario Ministry of Transportation, Ontario Structure Inspection Manual.

Masonry is made of natural stone blocks or clay bricks usually bonded together by mortar. Although not a common construction material today, masonry was used in retaining walls, abutments, piers or arches, primarily in the 19th century while brick masonry was only rarely used in highway structures. Types of masonry construction are Ashlar masonry, squared stones masonry and rubble masonry.

The following defects commonly occur in masonry:

- Cracking
1.1.5.1 Cracking

Cracks develop in masonry as a result of differential settlement of the structure, loss of mortar, thermal restraint and overloading leading to crushing and splitting of blocks.

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, as shown in Figure 1.1.5.1.

![Figure 1.1.5.1 Cracking through masonry](image)

1.1.5.2 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. Splitting, spalling and disintegration may also occur if adjacent blocks touch as a result of deformation of the arch ring.

1.1.5.3 Loss of mortar and stones

Loss of mortar is the result of the actions of water wash, plant growth or softening by water containing dissolved sulphates or chlorides. Partial disintegration of mortar may lead to loss of stone blocks.

Figure 1.1.5.3 shows evidence of loss of mortar.
1.1.5.3 loss of mortar.

1.1.5.4 Arch stones dropping

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 as shown in Figure 1.1.5.4.

1.1.5.5 Side wall movement at masonry arch

Excessive pressure normally due to heavy loads or vibration can cause the side walls of a masonry arch to move outwards away from the arch. This is a serious problem and will probably require a higher level inspection.

1.1.5.6 Deformation

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.

1.1.5.7 Separation of arch rings

Arches may comprise multiple rings or layers of bricks which combine in practice to form a single arch. The rings may delaminate if the mortar fails or if overloading or settlement occurs. The position and degree of separation should be recorded.
1.1.6 Protective coatings

Coating defects are not necessarily serious but they are indicative of potential weaknesses in the coating system and eventual loss of protection to the coated surface.

Protective coatings generally have a shorter life than the life of the structure. Hot-dipped galvanising may be totally lost in less than 30 years: even less in aggressive or abrasive conditions. Breakdown of paint or loss of galvanising is inevitable and should be anticipated. The rate of breakdown is dependent on a number of interrelated factors with the duration of continuous exposure to water being a significant factor. In addition to rainfall, exposure to water arises from immersion and from condensation and the effect may be increased if the moisture contains windborne salt and the salt is not removed by rain. Accumulation of debris, bird droppings, flaking paint, for example, will retain moisture and promote corrosion.

In addition to eventual failure of a coating system by weathering, premature failure may result from:

- poor adhesion due to incorrect specification, preparation or application of the coating
- incompatibility of successive coats
- rusting due to inadequate surface preparation and/or priming paint
- localised failure due to mechanical damage
- inadequate paint film thickness on sharp edges, welds and paint shadow areas.

Expert advice may be required to establish the cause of failure and recommend suitable remedial action.

1.1.7 Fibre reinforced polymer 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.

1.2 - Common causes of bridge deterioration

1.2.1 Concrete bridges

The following section lists the various types of reinforced and prestressed concrete bridges and lists the main problems associated with each type.
1.2.1.1 Monolithic and simply supported T-beams

The majority of monolithic structures are T-beam bridges with the whole structure cast-in-situ. Spans are generally small but bridges of this type may have up to 5 spans. This can cause significant strains at piers, columns and abutments due to temperature-related movements possibly leading to cracking and other relative displacement or distortion of the beam/wall joint at the abutment. Cracking may also occur at the column faces of the furthest pier from the centre of the bridge due to temperature-related movement. This type of structure may also exhibit cracking and staining of the underside of the deck in negative moment areas (near to the junction of the deck with pier/column and abutments).

T-beam bridges often have insufficient shear reinforcement near supports and diagonal shear cracking may be observed at 1/3rd of the span from the support. The abutments and wing-walls were frequently cast monolithically. Heavy cracking, spalling and movements may be observed at the wing-wall joints especially in the case of higher abutment walls.

Gravel fills were often placed over the deck of these structures with a sprayed seal which did not cover the full width of deck. Deck drainage was often poor allowing water ponding on the road surface which could then penetrate the concrete leading to efflorescence and spalling of the underside of deck.

Simply supported T-beam structures are generally a later design that features increased shear resistance in the beams and reduces risk of shear cracking. Some flexural cracking of the beams might be seen at mid-span especially on structures which carry frequent heavy loads. The beams were sometimes fixed at one end using a locating dowel with the other end free to move. The free end frequently locks with the consequence that the beam may crack and spall at both ends. The cross-head/bearing concrete at the beam supports can also spall due to frictional forces as the original debonding layer of grease or malthoid at the bearing surface deteriorates.

1.2.1.2 Precast I beams

The first precast I beams were made in 1949 and used on the Kiewa Valley Highway bridges. These beams were made from normal strength concrete and reinforcement. By the early 1960s, standard precast high strength reinforced and prestressed concrete beams were in use for spans of 9.1 to 18.3 metres (originally 30 to 60 feet). These beams have generally performed well over the years but some of the precast reinforced concrete beams have minor flexural cracking at mid-span.

NAASRA beam sections came into use in 1970 and were only adopted for long span structures in 1976. The NAASRA type 4 beams have been used for simply supported prestressed beams up to 33 metres, but also used for continuous prestressed beams of longer spans. This was accomplished by casting load bearing diaphragms at the piers which encased the ends of the beams.

The beams were also connected on the bottom bulb by heavy steel bars welded together. In recent years a new bulb tee section has been used in place of the type 4 NAASRA beam for spans up to 36.5 metres.

Prestressed beams can exhibit cracking at the ends in response to the prestressing forces in the strands. The cracks are normally horizontal and the result of inadequacies in reinforcement detailing in the end block. If the beam end is cast into a diaphragm these cracks are concealed and sealed against ingress of moisture. If cracking of this nature is discovered during an inspection, it must be reported. Skewed beam ends are vulnerable to spalling damage during production at the bottom surface and at the apex of the end. The damage occurs when stress is transferred into the beams. Exposed reinforcement is normally patched prior to delivery to site and the patches may be visible on inspection. Severely damaged beams may be rejected and are unlikely to be seen during inspections.

1.2.1.3 Precast prestressed inverted T beams

These beams were used during the 1970s to give a flat under-side to bridges crossing freeways. This was done for aesthetic reasons as the appearance is more appealing to the driver than the interrupted underside of a T beam bridge. Spans were usually in the range of 32 to 36 metres with the designs being continuous for live load. These beams were not an efficient section and lost favour with designers. No problems have been encountered with these types of structures.

1.2.1.4 Box-girder bridges

Box-girder bridges are generally cast-in-place and post-tensioned. A number of problems can occur during construction and at post-tensioning.
The major maintenance risk for this type of bridge is that the grout around the post tensioning tendons is incomplete and does not provide adequate protection against corrosion of the tendons.

Serious concerns have been identified in some overseas countries where de-icing salts are used on the road surface but to-date no evidence of tendon corrosion has been discovered in VicRoads bridges.

Some box-girders can be precast in segments and post-tensioned when erected in place. Bell Street Bridge over the Tullamarine Freeway and the West Gate Elevated Freeway being two structures of this type. Minor problems have occurred at Bell Street with slight moisture penetration of the joints between segments and cracking in the internal diaphragms due to high stress.

1.2.1.5 Prestressed voided flat slab bridges

Cast-in-place prestressed voided flat slab bridges provide an attractive shallow depth superstructure, ideal for very wide bridges and with spans in the range of 35 metres. Larger spans are relatively heavy and uneconomical although variable depth voided slabs of 40 metres have been built.

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 to-date.

1.2.1.6 Reinforced concrete flat slab bridges

This is a type of monolithic cast-in-place multi-span bridge, typically with 5 spans which 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 primarily due to thermally induced movements but also if the bridge is subject to the passage of large numbers of heavy vehicles.

The deck slab in this type of bridge often has a shrinkage crack which runs almost directly down the centreline of the slab. Provided this remains dry it is of no concern.

The final span is a short cantilever from the pier sometimes with a transverse beam stiffening the end of the deck. Vertical precast concrete wall units are placed against the stiffening beam at the end of the deck to retain the approach embankment fill. Spalling can occur due to friction between the wall units caused by vertical movement of the cantilever deck. Moisture may seep through the deck/wall joint. Movement of the wingwalls can occur in bridges with high abutments due to the correspondingly high fill pressures.

1.2.1.7 Rail in slab concrete bridges

These bridges comprise old railway lines spaced close together with a concrete deck cast on top with a layer of light mesh. Used on short spans with deck thickness of approximately 250mm, these superstructures are generally strong with good shear capacity provided by the rail heads and effective lateral load distribution by the plate effect. Under the effect of repetitive heavy loads they can deflect considerably leading to cracking of the concrete deck and loss of the bond between the rails and concrete. The condition of the deck concrete is vital to the stiffness and lateral load distribution it provides. The concrete acts as a shear key between the rails. Deterioration of the deck concrete is the cause of significant loss of capacity.

The railway lines can corrode with significant loss of section which may require repair. The parapet at the back of the footway is generally provided by deep I-beams which are often in direct contact with the footway soil backfill leading to the potential for severe corrosion to the hidden surface. Severe corrosion to this hidden surface may become evident on the external face of the beam web or the bottom flange.

Some bridges of this type have been successfully strengthened with the use of a reinforced concrete deck overlay. Use of an overlay in this manner is subject to accommodation of the thickened deck in the vertical grade of the road and consideration of adjacent property usage.

1.2.1.8 Precast prestressed slabs

Introduced in 1958 for spans of 4.6 to 9.1 metres (originally 15 to 30 feet) these units are held together by transverse tensioning rods in cored holes through the beam webs.

Slabs are 620mm wide, vary in depth between 160mm and 315mm and have a cast-insitu shear key between units. Although the transverse tensioning rods hold the decking firmly together, the shear-key concrete can crack and fragment allowing moisture penetration. The slabs have two layers of malthoid at the bearing surfaces...
under the ends of spans with a cast-in-place infill over the piers and abutments. Movement of the beams may cause cracking at the beam ends allowing moisture to penetrate to the crosshead.

The transverse tensioning rods require periodical checking and tightening. If they become loose, this reduces transverse load distribution in the deck leading to higher live loads on individual beams and the potential for long-term failure.

In 1961 a 10.7 metres (originally 35 foot) long prestressed slab was introduced. This had two voids and did not include transverse tensioning rods. Units were independent of each other but a 100mm thick composite reinforced deck was added to provide lateral load distribution between the units.

New South Wales (RTA) designed precast prestressed planks with spans of 7 to 15 metres were used in the late 1970s and 1980s with composite cast-in-place overlays ranging from 125mm to 145mm in thickness. These units had shear keys cast as part of the overlay. The overlay was made continuous over a maximum of three spans to reduce the number of expansion joints and improve the ride-quality over the structure. Some cracking problems have been experienced in the deck over the piers with this type of design. Examples of wide precast prestressed slab bridges have been built but these are now out of favour.

Early in 1993 a new prestressed slab, with a span of 17½ m, was introduced from Queensland. These units are designed for T44 live loading and plate action is achieved via transverse tension rods through the deck slabs. The units have voids with solid diaphragms at each transverse tension rod location. A sprayed seal is laid over the top of the slabs for anti-skid purposes.

1.2.1.9 Precast U slabs

1.2.1.9.1 General

1951 saw the introduction of precast reinforced concrete inverted U slabs using normal strength concrete and reinforcement. These units were designed for half the axle load of the design vehicle and acted independently of each other. Fill was usually placed on top of the U-slabs to enhance the lateral load distribution between units.

These bridges suffer from moisture penetration between the U-slab legs and also through the expansion joints at the ends of the spans. The beams themselves appear to be strong and only minor flexural cracking at mid-span is normally observed. The kerb slabs are a solid section and the kerb is precast with the beam. If shoulders are unsealed, moisture-related problems are usually most severe under the edge of the seal.

High-strength inverted U-slabs with shear keys and bolts between the vertical legs of adjoining slabs were adopted in 1962. An amended 1965 design increased reinforcement sizes, added mesh in the top flange and altered bolt positions. These U-slabs were designed for 0.47 of axle load of the design vehicle, relying on the shear keys and bolts to provide adequate lateral load distribution. These units had a number of problems including cracking at the top of the legs from over tightening of the bolts by constructors trying to bring the legs of the slabs together. Placing of the high strength concrete in the small shear keys was also a major problem with the joint concrete either setting too quickly or with shrinkage cracking of the joint concrete.

When subjected to repeat heavy loading, the concrete in the joint tends to crack and fragment allowing moisture to leak through the deck. The bolts also get loose due to vibration and the nuts fall off leading to a significant reduction in lateral load distribution between slabs and a consequential loss of load capacity. The slabs become overstressed (the proportion of axle load can rise to 0.67) with heavy flexural cracking of the legs at mid-span. Many of these bridges have now been strengthened by the use of a 140mm reinforced concrete overlay which extends over three spans.

In 1976, with the introduction of T44 loading, the shear keys and bolts between the legs were eliminated and all U-slab bridges had a 140mm high strength reinforced concrete overlay added during construction. The U-slab sections were decreased in depth for the span but reinforcement, both flexural and shear, was substantially increased.

A small number of prestressed concrete U-slab bridges were built, including double width U-slabs, but these were not successful due to levelling problems at the legs after stressing. With the introduction of the prestressed voided T slabs, use of stressed U-slabs was abandoned.

1.2.1.9.2 Inspection of Precast U slabs

(a) 1951 Standard Units
Deficiencies: The 1951 standard units were designed to act as individual units with a minimum depth of pavement material over them. Assessment of these units has indicated a theoretical deficiency in flexural and shear capacity.

Inspection and Monitoring: The legs of these units shall be inspected for flexural cracking, particularly near mid-span and for shear cracking near the supports as shown in Figure 1.2.1.9.2, side elevation.

These units should also be inspected for any cracks along the centreline or interface between the legs and the upper section. They should be inspected for any signs of cracking or spalling of the concrete from around the reinforcement in the bottom of the legs.

If there is evidence of cracking, the spacing, widths and patterns of cracking shall be recorded and photographed.

Some U-slabs are supported on mortar pads whilst others are supported on plain elastomeric bearing pads. There is a tendency for these mortar pads to crack and fail under repeated heavy loading. This in turn may lead to failure of the road pavement along the line of the interface between adjacent U-slabs and destroy any load transfer. Mortar pad and elastomeric bearing pad supports should be inspected and their condition recorded. Particular note should be made of situations where mortar pad failure has resulted in differential settlement of one or more U-slabs relative to adjacent slabs.

Evidence of water penetration such as efflorescence staining and stalactite growth along and between the soffits of the U-slab legs is an indicator of possible independent movement and loss of load sharing between adjacent slabs.

(b) 1962 Standard Units

Deficiencies: These units were designed to act in conjunction with adjacent units in supporting traffic loads. For this reason, the legs of adjacent units were bolted and cast in place; concrete shear keys were provided in the top corners of these units. However, under the action of repeated heavy loads which exceed the original design capacities, many of these bridges have suffered severe damage. This is particularly true of skew bridges where induced torsional loads in the units have not been explicitly designed for. The shear keys have fractured and the bolts have tended to become loose. If either of these actions have occurred, it is highly likely that reflective cracking would be evident in the deck bituminous surfacing.

Inspection and Monitoring: Refer to Figure 1.2.1.9.2. These units shall be inspected to determine whether any of the bolts are corroded, loose or broken. This shall be recorded and reported for maintenance purpose.

U-slabs shall also be inspected to determine whether there is any evidence of cracking of the shear keys. This shall be determined by viewing between the legs of adjacent units from beneath for direct signs of cracking or for any evidence of water leakage and by looking for evidence of reflective cracking in the deck bituminous surface, along the interfaces of the precast units. An inspection shall also be made from beneath the bridge to determine if there is any evidence of differential vertical movement between the legs of adjacent slabs under traffic loading. Such movement provides an indication of shear key concrete failure.

A thorough inspection shall also be carried out for the legs of the U-Slabs to determine whether there is any evidence of flexural or shear cracks. These units should also be inspected for any cracks along the centreline or interface between the legs and the upper section. They should be inspected for any signs of cracking or spalling of the concrete from around the reinforcement in the bottom of the legs. If any of the above signs of distress are evident, they shall be recorded in detail and photographed.

As with the 1950s U-slabs, some slabs are supported on mortar pads whilst others are supported on plain elastomeric bearing pads. There is a tendency for these mortar pads to crack and fail under repeated heavy loading. This in turn may lead to failure of the road pavement along the line of the interface between adjacent U-slabs and destroy any load transfer. Mortar pad and elastomeric bearing pad supports should be inspected and their condition recorded. Particular note should be made of situations where mortar pad failure has resulted in differential settlement of one or more U-slabs relative to adjacent slabs.
Figure 1.2.1.9.2 – 1962 SERIES U-SLAB BRIDGES

Areas to inspect for flexural cracking

Side Elevation

Inspect for crushing of cast-in-situ concrete shear keys and reflective cracking in bituminous surfacing

Cross Section

Inspect for loosening of bolts

Inspect for signs of water leakage through shear keys-efflorescence on sides and soffits of slabs
### 1.2.1.9.3 Rating of U-Slabs

#### Condition Rating

Once the inspection of the structure has been conducted, the following matrix (Figure 1.2.1.9.3.1) may be used to establish a relative rating of the U-slabs. The matrix combinations can then be used as part of the bridge monitoring process to give an indication of the structure condition.

<table>
<thead>
<tr>
<th></th>
<th>Road Structures Inspection Manual</th>
<th>Water Leakage/Staining #</th>
<th>Flexural Cracking</th>
<th>Shear Keys - Reflective cracking</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>in one span only</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>in more than one span</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>in all spans</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>in wheel path</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td></td>
</tr>
<tr>
<td>E</td>
<td>outer edge U-slabs</td>
<td>4</td>
<td>4</td>
<td>4</td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>in traffic lane</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td></td>
</tr>
<tr>
<td>G</td>
<td>shoulder of bridge</td>
<td>6</td>
<td>6</td>
<td>6</td>
<td></td>
</tr>
</tbody>
</table>

Figure 1.2.1.9.3.1 – Condition Rating Matrix

# Water leakage through the longitudinal shear keys

Structures subjected to repeated heavy loading particularly those on significant skews tend to deteriorate more rapidly and the shear keys and bolts are no longer able to distribute wheel loads to adjacent U-slabs. This usually results in flexural cracking of the beams and ultimately failure if not attended to.

The first sign of shear key deterioration is water penetrating through the deck and between the legs of adjacent units leaving efflorescence and staining on the legs of the beams. This generally occurs, initially along the wheel paths and/or at the outer edge U-slabs particularly beneath where the kerb and pavement meet or under unsealed shoulders where water is more likely to pond.

Water seepage from the expansion joint above the abutments and piers is also common however this should not be confused with water seepage between the U-slabs. This leakage occurs as a result of damage to the seal material of the expansion joints. Although this should be addressed as part of maintenance, it is not a direct indication of structural deterioration. If not attended to, it could potentially lead to corrosion of substructure elements.

Reflective cracking in the road pavement occurs once the shear keys begin to deteriorate, U-slab bolts are loose and there is differential movement between the U-slabs.

The rating of the U-slabs can be determined with the use of the manual (photos) and the findings from the inspection to determine the condition of the slabs which can be used to assess the urgency of the maintenance, strengthening or possible replacement.

Refer to Figure 1.2.1.9.3.2 (a,b,c) for examples of the different ratings for each category.
Figure 1.2.1.9.3.2 a - WATER LEAKAGE/STAINING

Minor

Moderate
Severe
Figure 1.2.1.9.3.2 b - FLEXURAL CRACKING

Minor

Moderate
Severe
Figure 1.2.1.9.3.2 c - SHEAR KEYS – REFLECTIVE CRACKING

Minor

Moderate
Severe
Example:
A monitoring inspection was conducted on a bridge constructed in 1963 with bolted high strength reinforced concrete U-slabs with no reinforced concrete overlay. The columns, abutments and crossheads are all cast insitu reinforced concrete.

Generally the piers and abutments were in good condition with only minor water seepage. When the soffit of the structure was inspected it was noted that minor water seepage was evident between the outer two U-slabs on the up and down stream side of the structure and between U-slabs 7 and 8 from the east side of the structure. Efflorescence and stalactites were also evident between the outer U-slabs with minor water staining and silt build up on the soffit of most of the U-slab legs. Severe flexural cracking was evident throughout the legs of the U-slabs with spacing between the flexural cracks ranging from 50 to 200 mm. From the top of the structure slight depressions in the road pavement were evident however longitudinal cracking was difficult to identify due to a recent spray seal. Shear keys were exposed near the kerbs and the scuppers were blocked.

From these findings in the Monitoring Inspection Report the following U-slab Rating was given.

<table>
<thead>
<tr>
<th>Water Leakage</th>
<th>Flexural Cracking</th>
<th>Shear Keys - Reflective cracking</th>
</tr>
</thead>
<tbody>
<tr>
<td>2CE</td>
<td>6C</td>
<td>5CD</td>
</tr>
</tbody>
</table>

**Bridge Details**
The following bridge details may provide an indication of the likelihood of damage occurring and probable rates of deterioration. They should be included in the assessment.

- bridge skew – 0°, 14°, 26° 30°, 36° 50°or 45°.
- smooth or rough approach road profile
- operational speed of road for heavy vehicles – 40, 60, 80, 100 km/hr.
- overall bridge width
- number of traffic lanes / width between kerbs
- single / multiple spans and span length
- mortar pads or neoprene strip bearing supports.

The greater the bridge skew, the higher the probability that the shear keys will crack and fracture under repeated loading.

Uneven approach road profiles increase the dynamic interaction between the bridge and heavy vehicles and potentially rapidly accelerating the structural degradation of the bridge.

The greater the speed of heavy vehicles, the greater the potential damage to the structure.

Greater overall bridge width for a given number of traffic lanes may lead to better load distribution and decrease in the probability of shear keys cracking and failing. This possible correlation between bridge width and damage requires further review based on the results of monitor inspections.

The relative effects of single versus multiple spans and span length on the probability of shear key cracking and rate of fracture are not known at this stage. Further review is required.

Elastomeric strip bearing supports tend to reduce the dynamic impact on the bridge superstructure and shear keys.

Other links between design detailing and structure performance may be established with further inspections and reviews.

**Traffic Volume, Mix and Speed**
The greater the AADT and number of trucks per day the greater the probability of damage to the shear keys and rapid rate of deterioration. The following table attempts to provide a measure of the relative risk associated with different traffic volumes.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Risk</th>
<th>Value</th>
<th>Risk</th>
<th>Value</th>
<th>Risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>AADT</td>
<td>&lt; 1000</td>
<td>Minor</td>
<td>1000 - 5000</td>
<td>Moderate</td>
<td>&gt; 5000</td>
<td>High</td>
</tr>
<tr>
<td>Trucks/day</td>
<td>&lt; 50</td>
<td>Minor</td>
<td>50 - 200</td>
<td>Moderate</td>
<td>&gt; 200</td>
<td>High</td>
</tr>
<tr>
<td>Speed km/hr</td>
<td>&lt; 60</td>
<td>Minor</td>
<td>60 - 80</td>
<td>Moderate</td>
<td>&gt; 80</td>
<td>High</td>
</tr>
</tbody>
</table>
1.2.1.10 Precast prestressed voided T slabs

These standard slabs span from 8 metres to 19 metres and were developed in 1986. The slabs vary in depth from 250mm to 750mm and have a 140mm overlay. Width of the top flange of the T-slab varies from 900 to 1500mm to suit the width of bridge. These slabs were used quite extensively and were then the cheapest and most popular type unit for spans up to 19 metres. Problems have occurred with high neoprene bearings placed on sloping crossheads beneath the T-slabs.

1.2.1.11 Decks and overlays

Reinforced concrete decks are usually cast-in-place over beams. The deck is then surfaced with either a sprayed seal or a 50 millimetre thick bituminous surfacing. Permanent or sacrificial formwork comprising thin precast concrete slabs is used to eliminate the need to remove the formwork after casting the deck particularly for bridges over highways and railway lines.

Concrete decks without surfacing were increased in depth by 12 millimetres to allow for wear by traffic. This practice was discontinued due to temperature cracking of the surface which allowed moisture to penetrate into the deck.

In order to provide composite action between beams and deck, longitudinal shear connectors (shear studs) or projecting bars are provided on the tops of beams which project into the deck. A bevelled concrete cap was cast between the deck and beams on many older bridges. Cracking of the cap can occur along the fillet line at the deck. Cracking coincident with the location of the stud or projecting bar connectors might also be visible. Unless severe, this cracking is not serious.

Many older concrete bridges had reinforced concrete decks with a gravel fill together with a sprayed seal of the whole or partial width of the deck. Deck drainage was often through the kerbs at the top of gravel level. This system did not drain the deck well and allowed a considerable amount of water into the deck fill. This acted as a reservoir allowing water to penetrate the concrete deck causing severe corrosion of reinforcement and spalling of the underneath of the deck.

Reinforced concrete overlays have been used extensively to strengthen U-slab bridges that are subject to repeated heavy loading. Overlays have also been used to simultaneously widen bridges and to seal old concrete decks against moisture ingress. By making the overlay continuous the number of expansion joints has also been considerably reduced. Bridge designs now include a concrete overlay over the precast units to achieve required design live load capacity.

1.2.1.12 Diaphragms

A diaphragm is a transverse beam at the end of the deck which connects the beams together and provides stiffness. In older structures this may be the full depth of the beams. In later structures it can be of the order of 200mm to 250mm in depth.

Diaphragms may also be found at mid-span or at the third points in the span to provide web stiffening and to assist with live load distribution between beams.

Precast I-beam bridges continuous for live load can feature a wide heavily reinforced load bearing diaphragm at the piers. This diaphragm is required to support the full superstructure loads and to transfer load back to the substructure.

All diaphragms should be checked for cracking and for separation from the embedded beam-ends.

1.2.1.13 Kerbs, footways, posts and railing

The majority of older concrete bridges have either narrow kerbs (sometimes tapered in cross section) or 810mm wide kerbs tapered in plan at their ends. These wider kerbs had a barrier facing and were positioned in front of the railing leading to a dangerous situation such as errant vehicles could aviate and land on top of the barrier rather than be safely redirected. Some bridges used precast reinforced concrete kerb sections which dowelled into solid kerb sections at the intermediate posts. Spalling of the dowelled areas of these kerbs is common. The drainage path of the deck is designed to pass under these precast units and flow over the outside edge of deck. In practice, the moisture and road debris is trapped under the kerb units and the area remains wet and vegetation can grow there. Water flowing over the deck edge causes extensive staining of the concrete.
Where footways are constructed on bridges they should be inspected for pedestrian safety to ensure that footways are level and free from holes and trips. Moisture can penetrate footway slabs and adequate drainage of the area under the footway is required. If drainage is inadequate, dampness penetrates the deck; weed growth and efflorescence can then develop under bridge deck.

A number of different forms of posts and railings have been used on 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 guardrails, rectangular rolled hollow steel posts and rails, and reinforced concrete New Jersey barriers and F-type barriers with steel posts with one or two steel rails on top.

Steel guardrail has been installed in front of the existing bridge railing on some bridges. Steel plate parapets have been installed on older rail overpasses to provide an impermeable barrier. These comprise flat steel plate shaped to a sloping profile over a steel post and rail framework.

Steel mesh is fixed to barriers on pedestrian bridges and should be inspected for damage and tightness of the attachment bolts.

In locations of possible salt spray, aluminium railing has been used with a steel tensioning cable under the top rail. This tensioning cable is wrapped in plastic but if incorrectly installed or exposed to the weather because of missing rail sleeves, the plastic will disintegrate exposing the steel cable and heavy corrosion will occur in a short time span.

Steel guardrail on bridge-approaches must be attached to and overlap the bridge endposts and possibly continue over the bridge. This will prevent vehicles from hitting the bridge approach rail and being redirected into the endposts. Current standards require that there is a smooth transition in stiffness between the bridge approach barriers and the bridge barrier and that the entire length of barrier is free from snagging points.

For some time it has been the practice on highly trafficked, high-speed roads to provide rigid reinforced concrete bridge approach barriers. These are also designed to give a smooth transition in stiffness.

1.2.1.14 Abutments

Abutments will generally be of the following type:

- Spill-through abutments using a reinforced concrete crosshead supported on driven precast concrete piles or a frame type with reinforced concrete columns supported by a footing below ground
- Wall type abutments either reinforced or mass concrete
- Wall type comprising columns and a crosshead with infill wall panels between the columns
- Masonry walls comprising stone blocks
- Sill beams on piles (to support the bridge loads) with a reinforced earth wall
- Bed logs or logs installed in pig-pen style to support the cross heads

Spill-through abutments are a common type and usually have little or no cracking of the crosshead other than possibly shrinkage cracks. Frame type crossheads are more highly stressed and some flexural cracking may be found at mid-span between the columns or over the columns. Loss of retaining fill in front, beneath and behind the crossheads is also a common problem which requires attention.

Cracking of piles has been reported in bridges where large movements of the embankment fill have occurred as the abutment fails and is prevented from rotating or sliding forwards by the superstructure. Severe cracking and distress may occur in beams and bearing pedestals. Movement joints can lock and the bridge beams become overstressed particularly during periods of high temperature. The fender walls will often crack if beams bear hard against them or if the deck puts pressure on the top of the wall. Keeper walls on the ends of cross-heads can crack particularly on bridges on a steep cross fall where beams bear against them.

On many older bridges the ends of the steel RSJs were cast into the fender walls for a short distance. This invariably causes heavy cracking and spalling of the fender walls due to differential movement or rotation of the abutment crosshead. The spalling can become quite severe with complete loss of the fender wall in some instances.

All instances of cracking where movement of the abutment is suspected must be investigated to identify the cause and the appropriate remediation measures.
Wall abutments are generally more resistant to differential movement and less likely to exhibit cracking. Abutment walls may exhibit full depth cracks as a result of early-thermal cracking. Mass concrete walls are generally small in height. They may suffer from rotation or sliding instability or in some instances loss of fill from around their foundations due to scour. Wall abutments comprising columns with crossheads and thin infill panels might crack from the effects of earth pressure and shrinkage.

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

Bridges may have reinforced concrete approach slabs which rest on top of the fender walls. These are installed to reduce live load earth pressures behind the abutments and to maintain a smooth transition onto and off the bridge for fast moving and heavy traffic thus reducing the potential for impact loads on the structure.

Bridges comprising timber superstructures on stone masonry abutment walls were once common. A number of them had the original timber superstructure replaced by a concrete superstructure. These walls must be inspected for cracking, a sign of settlement, especially in heavily loaded areas such as directly under beams. Bridges of this type may feature a reinforced concrete capping beam on top of the masonry wall to distribute concentrated loads from beams.

1.2.1.15 Piers

There are several types of bridge pier:

- piles (pile bents) or columns supporting a crosshead (single or multiple columns)
- wall piers of constant or variable thickness (some of which consist of columns with a crosshead with infill panels between the columns)
- mass concrete
- masonry

Concrete piers can be cast in stages with horizontal construction joints between the stages. Horizontal cracking may occur around the construction joints.

All pier types may have deficiencies in reinforcement and may suffer from cracking.

Older structures may have poor quality concrete which can be eroded by the action of flowing water, sand, pebbles and grit. This can significantly reduce the amount of cover to the steel reinforcement. Shotcreting (sprayed concrete) may have been used to reinstate the concrete surface and this in itself may be eroded over time.

1.2.2 Steel bridges

There are several forms of steel bridge:

- rolled Steel Joist (RSJ) and Universal Beam (UB) - pre-fabricated I-sections
- plate girder (I-section welded or built-up from plate steel)
- trough girder (open trough with sloping webs built-up from plate steel)
- box girder (closed section possibly with two or more cells - e.g. West Gate Bridge steel span)
- truss

Some timber bridges have been strengthened by incorporating rolled I-sections while preserving the timber members for aesthetic reasons.

Modern steel bridges normally comprise one of the above steel beam types with a composite reinforced concrete deck. Composite action with the deck slab significantly enhances the strength of the steel beam.

Steel beam bridges with composite reinforced concrete decks are used for longer span structures. Fabricated steel plate girders are more expensive than prestressed concrete beams and will require repainting several times during their life.
Steel superstructures may deflect substantially under load and vibrate leading to the risk of cracking of the reinforced concrete deck particularly in old structure. Moisture, corrosion and efflorescence will normally be seen at the cracks. Cyclic loading and vibration is a cause of fatigue in steel components and affects steel plates (including gusset plates in truss bridges), welds and bolts.

Steel beams may be galvanised or painted or painted over galvanising. Galvanising and painting are temporary coatings and may deteriorate or suffer from mechanical damage. All steel components must be checked for condition of the paintwork and corrosion. If no action is taken, severe corrosion may result in loss of section and perforation of plates.

Steel beam bridges have steel bracing frames at the supports and at intervals throughout the length of the bridge to provide stability in the temporary state and to prevent lateral buckling in permanent conditions. These components and their connections must be inspected in the same way as the main members.

Splice plates are used to connect beam webs and flanges. These may be riveted, bolted or welded. All welded connections, splices and stiffeners should be closely inspected for any signs of cracking of the weld or metal immediately adjacent to it. Progressive increase in crack length and width is a symptom of fatigue and is caused by cyclic loading. Position and size of cracks must be accurately recorded and reported.

Bolted and riveted connections require inspection to check whether all connections are intact and tight. Missing bolts and nuts may arise as a result of fatigue failure of the bolt shank. Loose bolts can be detected by cracks in the coating system, by permanent displacement or by relative movement of the connected components as vehicles cross the deck.

Signs of excessive wear at pinned joints in trusses or other movement joints should be recorded.

Surfaces at member connections should be clean and free from debris, dirt and moisture as these are cause for corrosion of connections and connecting members. Uncontrolled drainage through leaking deck joints will discharge onto the ends of beams, cross bracing and bearings leading to corrosion. Signs of this should be recorded and rectified. Similarly, accumulation of water within closed units (e.g. box girders), such as leakage or condensation will lead to deterioration of the protective coating and eventually corrosion.

Longitudinal girders and truss members should be inspected for signs of deformation. This may be evidence of buckling of the member caused by overloading or sign of inadequate bracing and must be reported.

Steel members (particularly those made with lightweight steel sections as in truss bridges) are susceptible to damage by vehicle impact which, if severe, can significantly reduce the load carrying capacity of the structure. Impact from a high vehicle may cause damage to truss members in through-girders and trusses. Truss bridges are particularly vulnerable to impact damage as the failure of a single member or connection can lead to collapse of the structure. Impact damage to steel bridge components must be reported as a matter of priority.

1.2.3 Timber bridges

1.2.3.1 Timber stringers

Timber stringers (beams/main members) may be either round (stripped of bark but otherwise in the natural state), hewn (cut to size with an axe or sharp blade) or sawn. Hewn or sawn stringers will not have any outer sapwood.

Pipe rot is the deterioration and loss of the central soft core of the timber leaving a hollow section and proportionally loss of strength. Calculations may be required to verify if the stringer still has adequate reserves of strength. Timber stringers should be inspected for pipe rot. This is normally done by drilling or coring at critical locations such as at mid-span and measuring the thickness of the remaining timber. Inspection should be done at points along the span if severe pipe rot is suspected.

The stringers should also be checked at their ends for splitting (many timber stringers have anti-split bolts at their ends to control splitting). Stringers should have full bearing on either corbels or corbel blocks. Stringers should be checked for end rot especially at abutments where moisture or water leakage may be present.

Splitting of timber stringers can affect their performance and working life considerably. Splitting generally occurs along the grain and, unless severe, is not significant unless moisture is penetrating into the splits. Spiking of the decking into the timber stringers can cause splitting at the top and, with the presence of moisture and vibration of the spikes under traffic, this leads to spike rot.
If the stringer is severely split in the vertical plane, heavy loads might widen the split causing premature failure. Fractures due to overloading and splits that start from the bearing area and travel diagonally across the timber grains towards the top of the stringer are the most dangerous splits. In both cases the stringers will require relieving or replacing, although steel banding may be used to control the diagonal splitting. In this case, it is possible to consider a load limit on the structure.

Other problems which may occur with timber stringers are the presence of rotting knot holes (particularly at mid-span) and sagging or excessive deflection of the stringer under live load due to poor lateral distribution of loading via the decking.

Termite infestation of stringers, together with the associated loss of section, can seriously affect the performance of timber stringers. Careful inspection is required to identify if there is evidence of their presence.

**1.2.3.2 Corbels and corbel blocks**

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

**1.2.3.3 Decking**

Timber decking can be of two types: cross-beams with longitudinal decking, or cross-decking with thin longitudinal running planks. The cross beams are generally used on more heavily trafficked roads and the planks are usually used on minor roads.

Timber cross-beams are usually spaced at 1.2 metre centres to support the long-decking and legal axle loads. These should be inspected for end rot, top rot, bulging at the top due to ingress of moisture, sagging at mid-span due to excessive span length, fracture and severe splitting. Severe splitting and top rot can often be caused by spiking of the decking. The effect of termite damage on small sections can be severe. Careful inspection is required to identify if there is evidence of termite.

Timber cross-beams (normally 225mm x 175 mm @ 1.2 m centres) normally extend across a minimum of three beams unless designed specifically for simple spans. They should be firmly bolted to the beams and all bolts should be regularly checked to ensure tightness.

Long-decking should be laid in long continuous lengths and span at least three cross-beams unless designed specifically for simple spans. It should be securely bolted to the cross-beams at each end and at alternate intermediate cross-beams. This is done to stop flexing of the long-decking under load and to reduce the risk that the bolts will pull through the ends of the long-decking planks. Mild steel angle cleats are commonly used to bolt the long-decking to the cross-beams. These offer a rigid point against which the bolts can be tightened. Mild steel plates can bend on tightening and the bolts can work loose.

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

As the timber shrinks and dries, gaps will form between the planks and action may be required to close up the gaps by inserting additional thin sections of plank. This is especially important on bridges used by cyclists.

Timber cross-decking is often used on low volume unclassified roads, and is not as rigid as the long decking described above. In many cases the cross-decking is only spiked to the spiking plank or timber stringer below. This type of decking generally becomes loose and requires continual tightening of bolts if they are used. Longer spikes are often used but this only compounds the splitting and spike rot. Timber running planks are usually supplied with cross-decked bridges. These planks aid load distribution to the cross-decking. The running planks are usually of a thin section being only 40mm to 50mm thick. They are usually only spiked and easily become loose. These planks tend to split easily requiring constant replacement and also form a moisture trap which hastens rot of the cross-decking below. Some bridges have fill or asphalt over the cross-decking, although it does offer improved load distribution, this is not generally successful as the surface becomes uneven and cracked due to cross-decking movement. Surfacing also tends to trap a reservoir of moisture which accelerates timber rot.

Steel trough decking has been used to replace timber long-decking on a number of timber bridges. The troughs are usually sprayed with tar on the inside then filled with premixed asphalt to a level of approximately 50 millimetres above the top of the trough sections which is then compacted by the action of traffic loads. The infill should be resurfaced every 2 to 5 years approximately after opening (depending on traffic volumes and loads) to re-establish the longitudinal grade and cross-falls. 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 cause corrosion in the steel trough. 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 spreading under load.

If this problem occurs, it will normally be reflected in the road surface above as irregularities or pot-holes in the infill or areas of severe cracking. These are signs that the trough sections are deflecting excessively under load or are not effectively held down to the cross-beams. A few early trough decking bridges used concrete in place of the premixed asphalt but this was unsuccessful due to the large relative movements of the steel trough sections and the concrete infill. Cracking of the thin concrete section above the trough allowed moisture to penetrate and corrode the trough sections.

The most popular timber deck replacement has been the use of Waldren precast reinforced concrete deck units. The units are 1.99m long and are cast to the width of bridge required. Ferrules are cast into the ribs of the units to allow for attachment to the RSJs with a thin neoprene strip separating the concrete and the steel to dampen the traffic loading. Steel guardrail is attached to the outsides of the units to provide an improved safety barrier compared to the old timber post and rail. Solid end units are installed at the ends of the bridge for live load impacts onto the bridge. Cracking problems have been encountered with this deck replacement option particularly if the neoprene strip between the slab and the supporting steel beam is missing. Other problems include rotation of the clips that are used to hold the slab in position.

Other deck replacement options include Transfloor reinforced concrete deck formwork slabs with a reinforced concrete overlay cast on top. The advantage of this deck replacement is that the deck is made composite with the RSJs via shear stud converters to greatly improve superstructure capacity.

Nail laminated pine decks have been used in the past and have generally performed well, treated timber without heartwood was used and the laminates are butted over a cross beam. Heartwood components are highly susceptible to rotting, requiring early replacement. A poorly drained deck allows moisture to penetrate the laminates which dissolves the timber preservative accelerating the rate of rotting. Heavily trafficked decks tend to cause separation of the laminates allowing moisture between them, corroding the nails that join the laminates.

Bridgewood laminated veneer sheets (a proprietary material) have been used in a small number of examples. The sheets must be firmly anchored to the beams and the joints and edges treated with a bituminous paint to prevent deterioration of the laminates.

Stress laminated decks have been used in a small number of bridges. Restressing of the transverse steel rods or strands is required from time to time together with further monitoring of the tendon force depending on the dimensional stability of the timber.

### 1.2.3.4 Kerbs, posts and railing

**Visual inspection of the kerb condition and bolted connections is required.** The kerbs must be firmly held in place to ensure the strength of the barrier support.

Endpoints are usually round timbers and suffer from settlement, splitting, sap rot, base rot, piping, and top rot due to weathering.

Intermediate posts are normally timber but can be mild steel channel or angle sections or occasionally old railway lines.

**Visual inspection should include bolting, paintwork and impact damage from vehicles.**

Timber rails were originally used on timber bridges but steel guardrail is now a common addition. Connections must be inspected for rigidity. Painting is provided for traffic safety reasons and must be inspected. Rotting and split timber rails will require replacement.

On short bridges that are occasionally over topped by floodwater guideposts may only be used without rails, or if the bridge is long and traffic speed high, then posts with a steel wire cable may be used. The steel wire cable should be taut and well anchored at its ends to retain any errant vehicle. The wire cable should also be on the trafficked side of the post to give lateral support to the wire in retaining an errant vehicle. The wire should pass either through the post or through a steel eyelet attached to the post, and not simply rest upon a steel support bracket. The wire cable should also be checked for corrosion.
1.2.3.5 Piles

Piles for timber bridges can be of two functional types, those used to take vertical loads and support crossheads and those which take moments such as wingwall piles or stream fender piles.

Rot is most likely to occur at or just below ground level, at normal water level (usually 300mm to 600mm below walings) or around areas where there are large numbers of bolt holes such as walings and cross-bracing.

Piles which take moments are particularly susceptible at ground or normal water level where the maximum stress and maximum risk of rot coincide. If pipe rot has been detected in these critical areas the extent of the rotting must be investigated to determine the length of repair or replacement.

Care must also be taken to determine the 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 below ground and is moving under load, a void will be seen around the pile and the pile will move as load is applied. If this occurs in water, ripples will be seen to emanate from the moving pile. In scoured areas the pile must be inspected higher up at the original ground level.

The loaded areas at piles tops must be visually checked for rot or splitting; especially splits originating from below the crossheads.

Timber piles may be infested by termites in many parts of the state. Termites can enter the piles to a depth of 300mm below ground but usually enter via splits in the timber above ground. Their presence can be detected by the presence of small covered runways in the splits or along the outside of the pile. They may also stick to the probe when testing the pile for rot. Termites create runways in the timber which can be detected when probing the test hole as if the scraper is passing through a series of thin timber sections.

Piles can wear away at ground level or at bed level due to the action of abrasive gravels or sands. The abrasive gravels occur in the mountainous regions and the wear can usually be seen. Abrasion by sands usually occurs at or near rivers estuaries and is due to sand movement with the tides. Pile diameters of structures in these locations should be checked by divers for loss of section.

Timber piles in marine situations can also suffer attack from Teredo although this is rare in cooler sea-water. This attack can occur anywhere between bed level and mean low tide level. Presence of Teredo can be detected by either sacrificial 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 5mm to 10mm diameter holes in the piles below water. Teredo will bore networks of tunnels in the timber and the damage may go completely unnoticed until the pile fails below water level. Early detection is vital. The use of Old Growth turpentine piles will deter Teredo attack.

1.2.3.6 Walings and crossbraces

Walings and cross-bracing should be visually checked to ensure that the piles are adequately stiffened and to provide a rigid structure to resist the action of the stream and possible debris and log impact. Walings are usually positioned 300mm to 600 millimetres above normal water level and give a good indication of the relative water level at the time of inspection. If the water level is higher than the walings then the timber piles should be reinspected when the level returns to normal. Walings can also be a good indication of whether scour or silting is occurring at the pier. The inspector must report the components below water level that could not be inspected.

1.2.3.7 Crossheads

Crossheads on timber bridges are usually comprise sawn timbers approximately 300mm x 150mm in section which should be visually inspected. However, some bridges comprise hewn timbers which must be checked for pipe rot. Inspection of crossheads should check for the following:

- Presence of termites
- Top rot due to the presence of wet fill
- Weathering or end rot
- Splitting
- Rot or separation of crossheads that are spliced at the centre pile
- Sagging (i.e. that the crossheads are not overloaded) where beams are not directly over the piles
- Settlement of piles leading to sagging of the crossheads
· Condition of loaded timber cantilevers
· That the crossheads are fully supported on the piles and are not reliant on bolting to transfer loads
· Bolting to ensure tightness.

1.2.3.8 Abutments

a) Bedlogs and props
Timber bridge abutments may comprise stacks of bedlogs; others may have props resting on a bedlog to form a relieving abutment in front of the original abutment.

Items to check and record if present:
· pipe rot in load bearing areas
· load bearing of the timber stringers or props on the bedlogs
· severe crushing of the bedlogs under load
· excessive splitting or end rot of the bedlogs
· leaning of the bedlogs.

A bedlog may be placed in front of the other bedlogs to support the fill on which the bedlogs bear. These bedlogs do not support the stringers but are still important in retaining the fill and preventing scour beneath the bearing bedlogs.

Suspect piles and abutments might be propped to supplement their vertical load capacity. These props usually bear on bedlogs or heavy sawn timbers. The props should be inspected for rot if they consist of round or hewn timber which still contains the heartwood. If the prop is a sawn timber, pipe rot will not occur. However, the condition of the end bearing support, connections to bedlogs, splitting etc. should be examined and noted. The prop must be securely attached to the stringer or relieving crosshead, and capable of taking the direct load. Props must be stable; if a prop is mis-aligned or leaning, this must be recorded and the prop must be re-positioned.

b) Abutment sheeting, fenderwalls, wing caps and wing planks
Abutment sheeting and fender-walls are main structural elements; wing planks and wing caps are primarily aesthetic elements.

Abutment sheeting can consist of timber planks or precast reinforced concrete units placed behind the piles to hold the embankment fill in place. These members should be checked for rotting, cracking, bulging and undermining by the stream.

Fender walls can consist of timber sheeting or precast reinforced concrete units. RSJs can also be used, in which case cast in-situ concrete is placed around the ends of the RSJs. These members should be checked for rotting, cracking, bulging or cracking/loss of concrete around the RSJs.

1.2.4 Deck joints

1.2.4.1 General
Purpose of deck joints is to seal the gap between the end of a bridge deck and the fender wall against the ingress of water and debris. Joints are designed to accommodate thermal and rotational movements in the bridge deck, normally by the use of a flexible seal in the gap. A number of different types of expansion joint have been used in the past.

Early bridges featured short spans and simple supports for which the required movement capacity of the joint was small. Materials with a small movement capacity such as cork, bituminous impregnated fibreboard, butyl impregnated polyurethane foam, styrene and foam strips were used. Asphalt, rubberised bitumen or polyurethane was often poured on top of the joint to seal it from moisture penetration. Many of these joints failed due to the joint material debonding or being inelastic. Sealant placed too high in the joint gap tended to crack and was lost.

As spans increased, so did the width of expansion joint, and compression seals were required to cater for the movements expected. Neoprene tube was the earliest recorded type of seal but proved to be inelastic and often fell through the joint leaving it completely open. Compression seals were then developed. These can be placed between concrete surfaces, steel angles, steel plates and proprietary ‘concrete’ headers/nosings. Compression
seals may debond and gradually move upwards to the top of the joint where traffic damages the seal or, in some cases, completely removes it. Steel angles are susceptible to impact loading from wheels, especially if dry packed mortar has been used beneath the angle. The mortar breaks up and the ensuing loss of support breaks the anchor bars holding the angle into the deck. The angles can then vibrate and move under load which cracks the bitumen at the edge of the angle.

A further type of expansion joint comprises a cellular neoprene seal attached to aluminium strips which in turn are bolted to the deck and abutment. These strips or rails may lift and break if the holding-down bolts are not properly secured into the underlying concrete. Holding-down bolts in cored holes are more vulnerable than bolts cast into the concrete although the latter type is at-risk if the concrete around the cast-in anchorage is not compacted correctly or if the holding-down bolts are not tensioned correctly. The seal and aluminium strips/rails may then be damaged possibly leading to a hazard for vehicles.

Steel finger plates and steel sliding plate joints have been used on larger span bridges. These joints may not incorporate a seal to prevent moisture penetration. Sliding plate joints can also vibrate loose causing a danger to traffic. These joint types were superseded by heavy duty rubber joints of the Transflex type comprising steel plates in an elastomeric plank. Debonding of the metal and rubber sections can occur in this type of joint and must be reported. Reinforced concrete nosings were used to support the joints but these can crack and fragment under repeated impact loads.

Asphaltic plug joints are used on bridge decks with small movements and sufficient asphalt cover. This joint consists of a 50mm thick (minimum) hot mix of selected aggregate and an elastomer modified bitumen binder and has the appearance of a strip (approximately 500mm wide) of dark asphalt. Defects may include fretting and loss of asphalt and movement of the steel cover plate (situated at the bottom of the joint under the asphaltic plug) where this has been used.

Cold-poured sealant joints are used mainly as replacement joints for bridges with a small range of movement. There have been examples of the use of this type of joint in new bridges in Victoria. This class of joint comprises a cold-poured sealant over a circular backing strip between proprietary concrete nosings/headers. The thickness of sealant is generally half the installation width of the joint. Joints of this type may fail by tearing or by debonding from the nosing leading to loss of the seal. Defects of this nature must be reported.

1.2.4.2 Inspection of Deck Joints

1.2.4.2.1 Procedure

Inspection of deck joints should be carried out as follows:

STEP 1 From the bridge construction drawings, determine the size, type, end treatment and any other relevant details of the deck joint(s). It should be noted that VicRoads practice is for the design drawings to show design criteria and 3 types of suitable joint for each bridge. That is, drawings show the type and size of joint, but the product details will appear on ‘as built’ drawings, when produced.

STEP 2 Prepare suitable summary sheets of data to be confirmed on site. The sheet should include an area for noting of any faults. Typical service faults are discussed below and a suitable data sheet is shown.

STEP 3 Carry out site inspection and record findings on the attached check list.

STEP 4 Recommend any necessary corrective action based on the results of inspection(s).

1.2.4.2.2 Joint defects

Most joint defects occur because installation has not been carried out in accordance with the manufacturer’s or Code requirements. For example, anchor bolts become loose because they were not installed and tightened correctly, or joints leak because the treatment at kerbs, medians or parapets is not in accordance with the design details.

Some common service faults are listed below: –

**Loose or missing anchor bolts**

This is the most common service fault, and is generally due to inadequate initial load in the bolt. Re-tightening of all bolts after a period of service is recommended. However, loose or missing bolts may be due to failure of the bolt or ferrule.

**Use of unsuitable anchor bolts**
All anchor bolts should be hexagon headed – use of socket head bolts such as “Unbrako’ is unsuitable because of the difficulties of re-tightening and replacement.

**Excessive water leakage**

It is likely that there will be some water leakage through most deck joints. It is important that leakage is minimised because it results in staining of the substructure, and deterioration of steelwork and concrete. All deck joints should be inspected from below the joint to assess the extent of leakage, and if possible to locate the cause of seal failure.

**Loose, damaged or missing seals**

Deck joint seals may be damaged or worn by vehicles or sharp objects such as stones, or may be poorly installed or of unsuitable material. Inspection should include examination along the seal in order to detect defects including loss of bond between lengths of seal.

**Excessive debris in joint**

Joints such as strip seal type should be designed to expel debris by the passage of vehicle tyres.

Joints having a deep recess such as strip seals with excessive ‘drape’ may be filled with debris especially adjacent to kerbs. Regular routine maintenance may consist of cleaning with compressed air or water, but the seals may need to be replaced to provide a long term solution.

**Joint width outside design movement range**

For good vehicle riding characteristics, the maximum width of expansion joint gap should be 70mm (measured square to the joint).

**Variation in joint width**

Deck joints should have uniform gap width, and variation in width along the joint may be due to poor initial installation, loose fittings or plan rotation of the bridge.

**Excessive noise**

Most deck joints cause some tyre noise, but loose components of joints such as sliding plate type can cause excessive noise. For this reason sliding plate joints are not recommended by the current Australian Bridge Design Code for vehicular traffic, but are used on footways to provide a smooth surface.

**Damage of joint nosing**

Where a concrete strip or ‘nosing’ is used adjacent to deck joints, inadequate reinforcement or compaction of the concrete may cause cracking or spalling of the nosing. These faults can result in unsafe driving conditions and should be repaired as soon as possible.

### 1.2.5 Step (Half) Joints

Step joint or half joint is a type of articulation in which a suspended beam or deck slab (the drop-in span) is supported on a short cantilever or corbel as shown in Figure 1.2.5. Step joints can be found on both concrete and steel structures on existing structures.

Step joints are normally positioned several metres away from the pier or abutment and often over live traffic or wide waterways. Traffic management, extensive scaffolding, machinery or custom designed suspended platforms are required to provide a safe access for personnel to inspect and maintain the joints.

This type of construction detail inherently leads to leakage through the joint. This causes debris and moisture to accumulate at the beam seats and eventual deterioration of the concrete and steel surfaces. Due to the difficulty in inspection and maintenance of step joints, VicRoads no longer permits the use of this type of construction detail.

Level 2 inspections shall identify the presence of step joints on the comments sheet. The inspector shall inspect step joints using binoculars unless the step joint is readily accessible. The inspection shall make comment and take photos on any staining, build up of debris, or cracking within 300mm of the stepped profile. Photos of the elevation of each side of the step joint and underside shall be taken.
1.2.6 Bearings

The following covers only the common types of bearing in past and present use. The first precast and cast in situ beam bridges sat on a layer of clear grease, a sheet of malthoid or in some cases a sheet of lead placed on the crosshead. Dowels projecting from the crosshead were used to locate beams but these have tended to break free from the ends of the concrete beams or, in some instances, the dowels have broken the top of the crosshead under the beam as a result of deck movement and edge loading.

Mortar pads were frequently used in the past and may sometimes be found in good condition although some mortar pads made by hand-ramming mortar into the gap under the beam have tended to crack and the mortar has spalled.

Steel base plates in conjunction with small steel bearing plates on the underside of beams have been used on a number of bridges. A phosphor bronze sliding plate was sometimes inserted between the steel plates to reduce friction.

Cast iron bearing blocks with sliding plates or pins, mild steel rollers and rocker bearings have also been used in conjunction with longer span steel beams. The performance of roller and rocker bearings can be adversely affected by grit and corrosion; bearings sometimes seize completely as a result of corrosion.

Large span, heavy concrete bridges such as box girders can be supported on pot or spherical bearings (bearings with a P.T.F.E. (Teflon) sliding disc). The P.T.F.E. strip can be squeezed out by vibration and its position should be recorded. Excessive rotation of the bearings should also be noted. On freeway bridges the pot bearings at the piers may be hidden by stainless steel skirts.

Elastomeric bearing are now in common use; either as a thin strip or pad usually 20mm thick, or in a rectangular form incorporating metal plates between the layers of elastomer. The thinner bearing strips/pads are normally used to support small span beams. However, if the bearing pedestals are poorly constructed then some parts of a pad may not carry load.

If poorly designed or manufactured, elastomeric bearings with steel plates can suffer from irregular bulging and shearing at the elastomer/metal plate interfaces. Elastomeric bearings rotate and deform in shear as the bridge moves and, in extreme cases, this can cause lift-off of the bearings at the edge, leading to over-stress of the opposite edge of the bearing. Irregular and uneven pedestal construction is a common problem associated with large bearings and can also lead to uneven development of stress in the bearings - over-stress in some locations and little or no stress in others.

Creep, shrinkage and elastic shortening due to post-tensioning can cause excessive shear stress on the bearings in box-girder bridges. Bearings may require resetting in this circumstance. This will require the beam to be
jacked-up - rarely done unless deformation is excessive. Actual deformation should be measured and reported if it is thought to be excessive.

Slippage (walking) of elastomeric bearings can occur, particularly in older structures where bearings retainers were not used. More recent designs (since the 1980s) incorporate bearing retainers that prevent slippage.

1.2.7 Culverts

1.2.7.1 Concrete box culverts

Early box-culverts were cast-in-place and many of these early examples suffer from corrosion of reinforcement, cracking and spalling due to lack of concrete cover, porous concrete and/or ingress of moisture. Once corrosion, cracking and spalling has commenced, progressive deterioration generally ensues.

Small precast inverted U culverts with precast concrete lids may exhibit significant cracking and spalling due to inadequate cover to reinforcement.

Larger precast concrete crown units have also been extensively used. Link slabs have been used between units in multi cell culverts to reduce construction costs and time.

The link slabs may be either precast in a casting yard, or cast on top of the culvert base slab and then lifted into position.

Box culverts are more susceptible to concrete problems under the edge of the seal if the shoulders are unsealed.

1.2.7.2 Concrete pipe culverts

Large pipe culverts have been used for many years. The pipes are susceptible to many defects that may arise from inadequacies in manufacture, handling, stacking, transportation and installation.

Large pipes may have a lifting hole. It is important that the hole be accurately positioned at the pipe obvert as elliptical reinforcement is used.

Pipe culverts should be inspected for the presence of cracks, spalls, line and level and stability of headwalls and wing walls.

1.2.7.3 Masonry arch culverts

Refer to clause 1.1.5 - Masonry.

1.2.7.4 Buried Corrugated Metal Structures (BCMS) - pipes and arch culverts

1.2.7.4.1 General

Large corrugated steel pipe and arch culverts have been used in Victoria for many years. A very limited number of corrugated aluminium culverts have also been used.

Unequal lateral soil pressure applied during construction, long term settlement or scour may lead to permanent deformations and instability of corrugated metal structures. These issues may also develop as a result of lack of thickness specified in the design process, loose, corroded or missing bolts, corrosion and loss of section of metal plates.

Significant deformations are potentially a high risk to the structural integrity and those who inspect and maintain them. Inspectors should not enter a corrugated metal pipe that has significant corrosion or deformation. A Level 3 investigation should be initiated as a matter of urgency.

Corrosion of steel culvert panels commences when the galvanised or other protective coating is damaged by impact or by abrasion resulting from the action of soil-particles in the flowing water or if the coating is lost through the normal sacrificial process. Contact with aggressive water or soils (natural ground or in the backfill) or other materials such as cattle droppings may also contribute to corrosion.

It may only be possible to assess the loss of metal in the water-side (open side) surface of the BCMS walls around the invert and within the splash-zone above water-level. There may be significant loss of metal thickness to the buried (soil-side) faces of the BCMS and, unless the metal becomes visibly perforated or so thin that it can be pierced with a hand pick or chisel point, it will not be possible to fully assess the degree of corrosion on the soil-side. If a detailed assessment of soil-side corrosion is required, this can be achieved by cutting samples from the wall-panels or by excavation to expose the BCMS. The extent and method of cutting or excavation must be agreed.
with the Principal Engineer - Structures in order to prevent the risk of de-stabilising the BCMS. It may also be possible to use non-destructive methods of testing such as ultra-sound to measure the thickness of the metal.

Issues that may require action to enable detailed inspection:
- the invert of the BCMS may be obscured by debris and or submerged below water-level
- if the BCMS is being used as a cattle underpass, the invert may be obscured by gravel and cattle-droppings
- if the BCMS is being used as a pedestrian underpass, the invert may be paved.

Features of a potentially unstable BCMS:
- the walls may be deformed
- the soffit may be propped
- the invert may be severely corroded and there may be significant loss of metal
- the backfill may be eroded or softened.

1.2.7.4.2 Inspection of Buried Corrugated Metal Structures

Confined Spaces Regulations
In all cases entry into BCMS must be managed in accordance with Confined Spaces Regulations together with VicRoads confined spaces procedures.

Unaccompanied Inspection
Unaccompanied inspection should not be attempted whatever the anticipated condition of the BCMS. The inspector should be accompanied by an assistant who is equipped with a mobile phone or some other means of summoning help in the event of an incident or emergency.

Level 1 Inspection Requirements
A severely corroded/damaged BCMS may be discovered for the first time during a Level 1 inspection. In this event prior to the undertaking the inspection a risk assessment should be undertaken to determine whether it is safe to enter. A BCMS showing signs of corrosion damage (section loss/delamination), deformation or if it has been propped (tommed) can be considered unsafe and should be referred to L2 bridge inspector or Asset Services for further inspection advice.

Further advice is provided in 1.2.7.4.3 on the procedure to be followed during inspections when there is debris obscuring the invert.

Risk Assessment
The inspector is advised to make a preliminary visit to the BCMS in order to assess the need for special means of access such as a ladder or the clearance of dense vegetation and to enable the preparation of a site-specific risk-assessment / Job-Safety Analysis. If a two-stage visit is impractical - due to the remoteness of the location for example - the inspector is advised to conduct a preliminary inspection and risk assessment on arrival at the BCMS and to continue with a detailed inspection only in the event that safe access is possible.

The inspection must be aborted if there is any safety concern that cannot be managed with the available equipment and a further visit should be planned only when all outstanding safety concerns have been satisfactorily resolved. Risk assessments must be in writing and must be recorded for future reference in RAS and in the Principal Engineer - Structures structure file.

1.2.7.4.3 Recommended Procedure for Inspection

Access to Records
Prior to the inspection of BCMS, the inspector or the manager responsible for the inspection is advised to review the most recent inspection reports and inform the inspection team of the last known condition of the BCMS together with any previously identified hazards.

In the normal course of events a Level 3 investigation will be arranged on the basis of a previously reported condition 3 or 4 rating and there will be a known history of deteriorating condition. Where a BCMS is already known to be severely corroded with significant loss of metal to the invert, or it is deformed or propped, a Safety Inspection (as described below) must be arranged. In these circumstances the possibility of instability must be assumed. However, a corrosion-damaged BCMS may be discovered for the first time during a Level 1, 2 or 3 inspection. With this in mind the procedure described below is to be adopted in accordance with the level of the inspection.

General Guidance Applicable to all Levels of Inspection
Adequate lighting must be available throughout the inspection;

In all cases an initial inspection should be conducted from outside of the BCMS to check for the presence of propping, visible severe corrosion, deformation and erosion of the surrounding soil;

If the planned inspection is at Level 1, 2 or 3 and any of these defects are present, the inspection must be aborted and a safety inspection, as described below, must be arranged;

Hand-tools (hammer, pick and chisel) must be available to take soundings and to probe the condition of the metal surfaces;

Under no circumstances whatsoever shall propping be removed during an inspection unless there is an alternative temporary works support system in place.

Notes

The inspection must be aborted if, at any stage, the inspector becomes concerned about safety;

Those managing inspections must ensure that if a potentially unstable BCMS is identified:

- Warning signs are erected promptly;
- A Safety Inspection is arranged at the earliest opportunity;
- The need for temporary closure is addressed promptly.

Level 1 Inspection

The fundamental principle is that an inspection at this level must only be attempted if the BCMS is in condition 1 or 2 and there is no propping or deformation.

- the BCMS is to be inspected from outside and, if the metal is rusting and has holes or there are missing sections or the BCMS is propped or there is visible deformation, the inspection must be terminated immediately and a report made to the appropriate manager

- if there is debris or other material obscuring the invert, the first 0.3 to 1m of the metal surface should be exposed without entering the BCMS (i.e. use a shovel standing outside BCMS) and its condition assessed as before

- if metal in the exposed area is badly rusted and there are holes or metal is missing, the inspection must be terminated and a report made to the appropriate manager as before

- if the Level 1 inspection team considers the BCMS to be unsafe, they must place signs warning the public and other workers of the potential hazard at both ends of the BCMS

- if invert metal is clearly visible and is sound and in good condition, clearing of debris may continue in 0.3 to 0.5m steps unless the condition of the corrugated metal deteriorates and holes or missing areas become visible - in which case the inspection must be terminated immediately and a report made to the appropriate manager.

Level 2 Inspection

The Level 2 inspection shall be as the Level 1 inspection described above with the following additions:

- provided that it is safe to do so the Level 2 inspector may probe the exposed metal surface from outside the BCMS and take soundings with a hammer and / or probe the metal with a pick or chisel point to assess the condition rating;

- if the visible part of the BCMS is in condition 1 or 2 the Level 2 inspector may enter the BCMS and continue to assess its condition;

- if the BCMS deteriorates to condition 3 or 4, the inspection must be aborted and a report must be made to the appropriate manager. It is recommended that a Safety Inspection is then arranged;
Level 3 Inspection

The Level 3 inspection shall be as the Level 2 inspection above with the following additions:

- The inspector must firstly assess the stability of the BCMS. Then and, only if it is safe to do so, the inspector may continue to inspect and quantify the condition of the BCMS and authorise removal of debris and other works to facilitate the completion of the inspection;

Safety Inspection;

The Safety Inspection shall be as the Level 3 inspection above with the following additions:

- the inspector may recommend further and more detailed investigation
- where the BCMS is used by the public (pedestrian and agricultural use for example), the inspector may recommend its temporary closure in the interests of safety
- the inspector may if they consider that it is safe to do so, authorise access for maintenance and repair work and determine the requirement for propping
- the propping system shall be designed by an engineer who is experienced in the design of temporary works and who shall either be:
  - qualified to Proof Engineer level in accordance with the VicRoads pre-qualification scheme; or
  - an engineer who meets with the approval of the Principal Engineer Structures.

1.2.7.4.4 Actions

Warning Signs

In the event that a serious defect is discovered and it is considered to be too dangerous for the inspection to continue, warning signs should be erected immediately at both ends of the BCMS in order to inform both the public and other VicRoads personnel of the hazard.

Stability of the Highway

In the event that the inspector considers the BCMS to be unstable and that there is a potential risk to users of the highway over the BCMS, the inspector must advise the Regional Director immediately.

Barricading of Unstable BCMS

If the inspector considers that a pedestrian underpass or a stock crossing (or any BCMS that is used for these purposes whether formally or informally) is considered to be unsafe for entry, this must be reported to the Regional Director immediately with the recommendation that the underpass is closed pending further investigation and repair. The following should be considered prior to closure:

- Consultation with users of the underpass and other affected parties (Police, catchment management authority and landowners for example);
- The route and signing of safe temporary diversions;
- The choice of barricade particularly where the BCMS carries water;

Register of Occurrences and Actions

In order to ensure that information is properly recorded for future reference, the structure record in RAS and in the Principal Engineer - Structures structure file should be updated with details of propping and any serious defect that is discovered such as deformation or severe corrosion damage together with details of any preventative action that is taken such as signing and barricading.

1.2.8 Causes of deterioration not related to bridge materials

The following items (which are not caused by defective materials) must be inspected and maintained in order to avoid structural deterioration:

1.2.8.1 Damage due to accidents

Vehicular impact damage is usually self evident and may be widespread, affecting all parts of a structure. In extreme cases the impacted item may be displaced or even dislodged. Concrete beam and slab bridges have been partially severed following impact from high loads.
Steel beams are particularly susceptible to impact from over-height vehicles which can cause severe deformation to the bottom flange and web of the beam and severely reduce its capacity. A technique for straightening and restoring impact-damaged steel beams using controlled application of heat is available.

Concrete components may crack, spall and fracture following impact, requiring extensive repair or replacement.

Pier columns and pile caps of bridges over navigable waterways may also be damaged as a result of impact by water vessels.

Structural damage or and/or displacement of the affected members will require a Level 3 investigation to assess the safety, structural stability and capacity of the bridge.

Other minor damage may occur such as abrasion and spalling of concrete which can result in eventual corrosion of reinforcement.

1.2.8.2 Drainage
Inadequate or impaired drainage may affect a bridge in several ways:
- flooding of the bridge deck which may create a serious traffic hazard
- water flowing over concrete, steel surfaces or bearings may result in corrosion or impaired performance of bearings
- build-up of debris retains moisture and promotes corrosion
- uncontrolled discharge from the deck can cause erosion of approaches, batters and possibly undermine foundations
- leakage from the bridge deck through joints and cracks can cause unsightly staining of beams, piers and abutments

Inadequate road-surface drainage from the bridge approaches can also cause erosion, piping and washout or scour of the approach embankment and batter slopes, particularly in areas where flows are concentrated at the ends of bridges, near the end post and at ends of kerbs or service ducts. These areas should be inspected particularly after heavy rain or flooding.

1.2.8.3 Debris
Build up of debris on the upstream side and on the deck of a bridge can cause the following adverse effects:
- very high imposed loads on the bridge possibly exceeding the design load
- impact loads particularly on slender piles leading to breakage of pile bents and/or total loss of piles
- blockage (partial or total) of the waterway which can cause flooding upstream, exacerbate problems of scour, undermine foundations and in extreme cases result in diversion of the watercourse

Build up of debris is usually most severe in bridges with small openings or low freeboard.

1.2.8.4 Vegetation
Uncontrolled and excessive growth of vegetation under or adjacent to bridges does not necessarily cause damage to the bridge. However, penetration of roots into the joints in a masonry or brick structure can cause damage and deterioration. Vegetation can result in a fire hazard, blockage of the waterway together with the build-up of debris and moisture around abutments and bearings. Presence of vegetation should be reported.

1.2.8.5 Scouring of foundations
Scour of river-bed material under and around foundations caused by stream flows or changes in the alignment of the stream channel can result in progressive settlement or movement of abutments and piers, which if not rectified may ultimately cause total failure of the bridge. In extreme cases, scour can completely remove the sedimentary material surrounding piles and has caused the failure of bridge superstructures leading to the need for replacement. Bridges on pile bents where the pile to cross-head connection is pinned are most vulnerable.
Aggradation is the process of deposition of river-bed materials that have been eroded upstream. Materials can be fine-graded silts and clays or coarser gravels and rocks depending on the nature of the upstream river-bed. Accumulation of material against or under a bridge has the following possible effects:

- high imposed loads on the bridge sub-structure and superstructure leading to damage or failure
- loss of waterway area leading to the risk of upstream flooding.

Where evidence of scour or aggradation of the stream bed is observed during an inspection, this shall be noted by the inspector for comparison with past and future inspections.

Beaching may be damaged and removed by scour action.

Adequacy of batter protection to abutments and the stream bed adjacent to pier foundations shall be noted together with changes in stream-bed condition upstream and downstream of the bridge.

1.2.8.6 Movement of the structure

Settlement and horizontal displacement of piers and abutments can be caused by:

- Scour
- Land slips around or under the bridge
- Earth pressure resulting from long-term settlement or movement of the embankment fill and underlying ground
- Collisions with vehicles and vessels
- Locking of bearings or expansion joints

Visual indicators of settlement and horizontal displacement:

- Closure or excessive opening of expansion joints
- Contact between the superstructure and abutment fender wall with associated cracking and spalling
- Steps in horizontal and vertical alignment of the road surface and bridge barriers, particularly at movement joints
- Cracking of abutments and columns or piers
- Cracking or excessive settlement of the approach embankments and in beaching or heave at the embankment toe
- Scour causing undermining of the foundations
- Out of verticality of adjacent roadside columns, poles and fences etc.

Observations of these indicators are an important aid in determining whether movement is continuing, seasonal or has ceased. Movements of this nature can continue over a long period of time and the ability to make comparisons with past inspections is useful in understanding the cause(s) of the movement and the appropriate response.

1.2.8.7 Condition of approach embankments

Embankments provide a smooth and navigable transition between the road level on approaches and the road level over the bridge. Embankments may also provide horizontal and vertical support to bridge abutments.

If excessive settlement of the approach embankments occurs immediately adjacent to a bridge abutment, this can cause poor ride-quality, possible damage to bridge joints, and high dynamic loads on the bridge deck.

Settlement of approach embankments is caused by inadequately compacted embankment fill and/or longer-term consolidation settlement of the underlying natural ground.

Other embankment defects commonly encountered are erosion, piping, washout and scour, particularly after heavy rain or flooding or a result of inadequate or blocked drainage.

1.2.9 Deterioration of roadside structures

Cantilever sign and high-mast lighting structures share some common features and vulnerabilities:
Fatigue-related failure of holding down bolts and to a lesser degree fatigue-related failure of the column and welds above the base-plate

Cracked, incomplete or missing mortar or grout under base-plates

Foundation-related failure leading to tilting of the structure

Fatigue failure is the result of rhythmic, wind-induced oscillation of the structure that occurs at relatively low wind speeds.

Truck-induced gusts also cause oscillation of sign and light structures and contribute to fatigue. Structural steels and other metals are susceptible to fatigue failure when subjected to cyclic loading. The degree of susceptibility depends on the ductility of the metal and its fracture toughness. Higher strength steels are generally more brittle than low strength steels and more prone to develop fatigue failure. Fatigue failure results in the development of cracking of components and eventually to their failure if the crack penetrates the full depth of the section.

1.2.9.1 Major sign structures

Large cantilever signs, butterfly signs and sign gantries must be inspected in order to detect fatigue-related failure (particularly in holding-down bolts) which could lead to collapse of the structure onto trafficked lanes. Columns of cantilever structures can progressively tilt over as a result of poor ground conditions and/or inadequate foundations.

These structures must be inspected for missing nuts, cracked bolts, loose connections, gaps between plates, cracked welds, heavy corrosion or small cracks and splits or ruptures of the columns and stiffeners, tilting columns. The concrete footings should also be inspected for cracking or spalling around the bolts or base plate, including signs of crushed or missing mortar below the base plate which could indicate that movement has occurred.

Nuts on holding-down bolts are normally 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.

1.2.9.2 High mast lighting

High mast lighting columns have a minimum height of 17m. High mast lighting must be inspected in order to detect fatigue-related failure (particularly in holding-down bolts) which could lead to collapse of the structure onto trafficked lanes.

These structures must be inspected for missing nuts, cracked bolts, loose connections, gaps between plates, cracked welds, heavy corrosion or small cracks and splits or ruptures of the columns and stiffeners, tilting columns. The concrete footings should also be inspected for cracking or spalling around the bolts or base plate, including signs of crushed or missing mortar below the base plate which could indicate that movement has occurred.

Nuts on holding-down bolts are normally 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.

1.2.9.3 Noise attenuation and visual screen walls

Noise attenuation and visual screen walls are normally located along major roads where there are residential or light commercial developments at the right of way boundary. They are commonly made from a range of materials including timber (plywood), concrete, steel, aluminium, acrylic and polycarbonate materials.

Some noise attenuation walls have built-in lighting powered by solar panels. These are subject to electrical problems and to vandalism.

Visual screen walls are similar to noise attenuation walls but are normally used to shield unattractive commercial or industrial development along important roads or shared use pedestrians routes.

Noise attenuation and visual screen walls must be inspected in order to prevent collapse onto trafficked carriageways or pedestrian ways.

Problems can occur due to:

- Rot and termites in timber particularly at or just below ground level
• Corrosion in steel and cracked welds
• Reinforcement corrosion
• Cracking and spalling concrete
• Collapse of panels due to failure of fixings including self-tapping screws
• Cracking of concrete and plastic panels
• Graffiti

The inspector should also observe and record problems associated with ground movement that may cause columns to move and allow panels to fall out. The footings should be inspected for cracking or spalling around the cast-in-situ or bolted connections.

1.2.9.4 Retaining walls

Retaining walls are generally made from timber, concrete, masonry and steel materials.

Problems can occur with the foundations of the wall due to:
• rot and termites in timber particularly at or just below ground level
• corrosion in steel and cracked welds
• reinforcement corrosion
• cracking and spalling concrete
• cracking of mortar or stone degradation in masonry walls
• settlement, sliding or overturning of the wall
• insufficient or ineffective weep holes to relieve pore pressure behind the wall

The inspector should also observe and record problems associated with ground movement that may be exerting unusual pressure on the walls.

1.2.9.5 Emergency median barrier access gates

Emergency median barrier access gates are generally made of galvanised steel.

There are currently two approved products for new installations which include ArmorGuard Gate System and BarrierGuard 800 Steel Gate. There are also legacy installations IronMan Median Gate which is no longer approved for new installations.

Problems with legacy installations include:
• difficulty removing locking pins
• difficulty manually moving gates and hinge side plates
• debris build up under rubber skirt (can be permanently removed).
Given the location of emergency median barrier access gates, it is more convenient and efficient that these structures are inspected, maintained, and tested at the same time under appropriate traffic management.

Regular maintenance of these structures in accordance to the supplier’s requirements is vital to ensure they perform in the event of an emergency. Maintenance may include cleaning debris, lubricating movable parts, and treating rust which may make their operation difficult when needed.